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Weir Sand-Bypassing Systems

by

J. Richard Weggel

SPECIAL REPORT NO. 8
APRIL 1981

U.S. ARMY, CORPS OF ENGINEERS
COASTAL ENGINEERING
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This report presents methodology for designing weir sand-bypassing systems. Jetties are generally shore-normal structures built at tidal inlets to fix the location of the inlet and associated navigation channel. In a weir-jetty system one or both jetties is constructed with a low weir section across which sediment can move into a deposition basin. The sediment caught in the excavated deposition basin is periodically dredged and bypassed to
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downdrift beaches to preclude serious erosion of those beaches. The design of a weir bypassing system requires knowledge of the wave and sand transport conditions at a site and involves locating and proportioning the jetties, weir section, deposition basin, and navigation channel, as well as selecting and designing the desired updrift and downdrift beach configuration.

Methods of data analysis and interpretation for weir-system design are presented along with guidance on proportioning the various components of a weir bypassing system.

PREFACE

This report is published to provide coastal engineers with guidelines for the design of weir-jetty sand-bypassing systems. A weir jetty is only one of several bypassing schemes that merits consideration when there is concern about erosion downdrift of a jetty project. The report does not intend to suggest that weir jetties are the best solution to all bypassing problems but simply presents a series of design goals and "rational" procedures to help evaluate a weir-jetty system against those design goals. It should provide some design techniques that will allow the designer to make judgmental decisions regarding such factors as weir height, length, orientation, etc. Since the design of a bypassing system depends critically on local conditions, absolute values for these variables cannot be set. The work was carried out under the coastal structures program of the U.S. Army Coastal Engineering Research Center (CERC).

This report is one of a series of reports to be published to form a Coastal Engineering Manual.

The report was prepared by Dr. J. Richard Weggel, Chief, Evaluation Branch, under the general supervision of N. Parker, Chief, Engineering Development Division. Many of the ideas or concepts expressed in the report did not originate with the author but were gleaned from discussions with other coastal engineers. For example, Dean M.P. O'Brien (1976) originated the concept of how an ideal weir-jetty system should perform so that it bypasses only the net longshore transport, and Dr. R. Dean developed the idea that the quantity of sand that needs to be stored in the updrift beach fillet depends on the timing and duration of transport reversals. Some of the concepts have been extended by the author and analysis methods suggested; any erroneous interpretation of the original concepts lies wholly with the author.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

Ted E. Bishop

TED E. BISHOP
Colonel, Corps of Engineers
Commander and Director

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To obtain
inches	25.4	millimeters
	2.54	centimeters
square inches	6.452	square centimeters
cubic inches	16.39	cubic centimeters
feet	30.48	centimeters
	0.3048	meters
square feet	0.0929	square meters
cubic feet	0.0283	cubic meters
yards	0.9144	meters
square yards	0.836	square meters
cubic yards	0.7646	cubic meters
miles	1.6093	kilometers
square miles	259.0	hectares
knots	1.852	kilometers per hour
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
millibars	1.0197×10^{-3}	kilograms per square centimeter
ounces	28.35	grams
pounds	453.6	grams
	0.4536	kilograms
ton, long	1.0160	metric tons
ton, short	0.9072	metric tons
degrees (angle)	0.01745	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[1]

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: $C = (5/9)(F - 32)$.

To obtain Kelvin (K) readings, use formula: $K = (5/9)(F - 32) + 273.15$.

SYMBOLS AND DEFINITIONS

a	empirical coefficient used in equation for runup, equal to 0.692 for a rubble structure
B	crest width of the structure
B_E	quantity of sand bypassed to the east from an inlet bar
B_W	quantity of sand bypassed to the west from an inlet bar
b	empirical coefficient used in expression for runup, equal to 0.504 for a rubble structure
C	weir discharge coefficient (assumed constant = 0.6)
d_s	water depth at the toe of a structure
$E\left(\frac{x}{\sqrt{4t\varepsilon/k}}\right)$	the error function complement of the argument $\frac{x}{\sqrt{4t\varepsilon/k}}$
e	base of the Naperian logarithms = 2.7182 . . .
g	*acceleration of gravity*
H_b	breaking wave height
H_i	incident wave height
H_o'	unrefracted deepwater wave height
H_t	transmitted wave height
h	height of structure crest above bottom
h_1	upstream height of water surface above weir crest
h_2	downstream height of water level above crest
I	immersed weight longshore sediment transport rate
K	Keulegan's K
K_o	wave transmission coefficient for waves transmitted by overtopping
K_t	wave transmission coefficient for waves propagated through a structure
L	weir length
L_o	deepwater wavelength

ℓ	jetty length •weir crest length
$P_{\ell s}$	longshore wave energy flux factor
Q_E	easterly volumetric longshore transport rate
Q_{gross}	gross volumetric longshore transport rate; total amount of sand passing a point on the beach during a specified interval
Q_{in}	volumetric sediment inflow rate to a control volume
Q_L	total longshore sediment transport to the left for an observer looking seaward
Q_ℓ	volumetric longshore sand transport rate
$Q_{\ell o}$	volumetric longshore sand transport rate along a straight beach (sand-bypassing rate around a groin when $t = \infty$; Pelnard-Considere model)
Q_{net}	volumetric net longshore transport rate
Q_o^*	empirical coefficient used in equation to determine wave overtopping rates
Q_{out}	volumetric sediment outflow rate from a control volume
Q_R	total longshore sediment transport to the right for an observer looking seaward
q	instantaneous longshore sand transport rate at a point on the beach •discharge of water over weir per unit of weir crest length •wave overtopping rate per unit structure length for waves approaching perpendicular to a structure
q_{net}	average value of the longshore sand transport rate at a point on the beach
q'	reduced wave overtopping rate per unit length of structure for waves approaching at an angle
R	runup height above the SWL (runup that would occur on a structure if the structure crest were above the limit of wave runup)
T	wave period
t	time
t_n	time after n intervals
t_o	initial time

t' time required following construction until shoreline at a groin or jetty reaches the end of the structure (time when sand bypassing begins)

x distance measured generally along the shore

y_s distance to shoreline measured from base line (a function of x)

α the angle between an incident wave ray and the weir axis
•empirical coefficient used in equation to determine wave overtopping rates

α_b angle a breaking wave crest makes with the shoreline

β empirical coefficient used in Goda's equation to predict wave transmission by overtopping

Δt time interval

$\Delta \forall$ change of sediment volume

$\Delta \forall_B$ sediment volume change on Bogue Banks, North Carolina

$\Delta \forall_S$ sediment volume change on Shackleford Banks, North Carolina

ε surf parameter given by $\varepsilon = \dfrac{\tan \Theta}{\sqrt{H_i / L_o'}}$.

$\dfrac{\varepsilon}{k}$ factor in the argument of the error function complement $E\left(\dfrac{x}{\sqrt{4t\varepsilon/k}}\right)$

θ angle the seaward face of a rubble-weir section makes with a horizontal

π 3.1415 . . .

T tidal period

τ time (interval of integration)

WEIR SAND-BYPASSING SYSTEMS

by
J. Richard Weggel

I. INTRODUCTION

The construction of jetties to provide safe navigation conditions at harbors or tidal inlets along sandy coasts usually results in interruption of the natural longshore transport of sand at the harbor or inlet. Sand that previously found its way from an inlet's updrift side to its downdrift side through natural processes is trapped in the updrift fillet or is diverted offshore. The resulting starvation of the downdrift beach can cause serious erosion unless measures are taken to transfer or bypass sand from the updrift side to the downdrift beaches. Several sand-bypassing methods used in the United States are discussed in Section 6.5 of the Shore Protection Manual (SPM) (U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1977).

The simplest method in concept, but in some respects the most difficult to implement, is to remove the sand accumulated in the fillet of the updrift jetty with a pipeline dredge and transfer it to the downdrift beach (Fig. 1). However, the dredge may be difficult to operate in an area exposed to ocean waves. This difficulty led to the development of fixed sand-bypassing plants. These plants are usually constructed on the updrift jetty and are partially protected from extremely large waves by the shallowness of the water in front of them. Fixed bypassing plants are usually limited in the amount of sand they can intercept and handle because of their lack of mobility. Another method for bypassing sand from an updrift fillet is by use of a conventional floating pipeline dredge to cut into the fillet from the ocean (described in Section 6.521 of the SPM). The dredge operates within a lagoon in the fillet, having closed the entrance channel behind it. After bypassing enough material, the dredge again cuts a channel to the ocean and exits from the lagoon. Experience in using this technique has been limited to a single instance at Port Hueneme, California.

At some harbors along an open coast with a shore-connected breakwater, bypassing is performed by dredging the shoal that accumulates at the distal end of the breakwater (Fig. 2). A dredge can usually operate in the quiet water behind the shoal itself and move into the harbor if adverse weather threatens. An example of this bypassing scheme is at Santa Barbara Harbor, California.

Shore-parallel offshore breakwaters built updrift of inlets or harbor entrances have also been used to establish a sheltered deposition area where a dredge can operate to bypass sand (Fig. 3). In some cases, such as at Channel Islands Harbor, California, the breakwater also serves to protect the harbor entrance. The bypassing system at Channel Islands Harbor is described in Section 6.522 of the SPM.

This report discusses the sand-bypassing system that requires the construction of a weir jetty. A typical weir-jetty system is shown in Figure 4. In this system, a part of the updrift jetty is depressed to form a weir section across which sand is transported to a deposition area by waves and tidal currents. A conventional pipeline dredge operating in the deposition

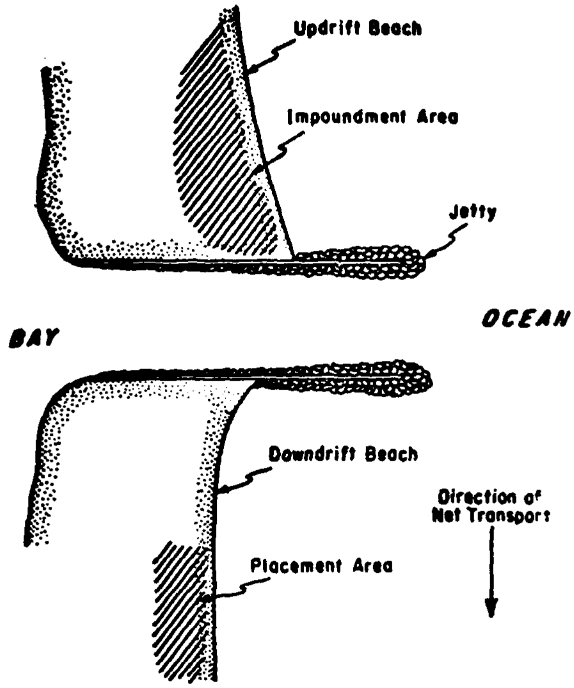

Figure 1. Sand impounded in updrift beach adjacent to jetty.

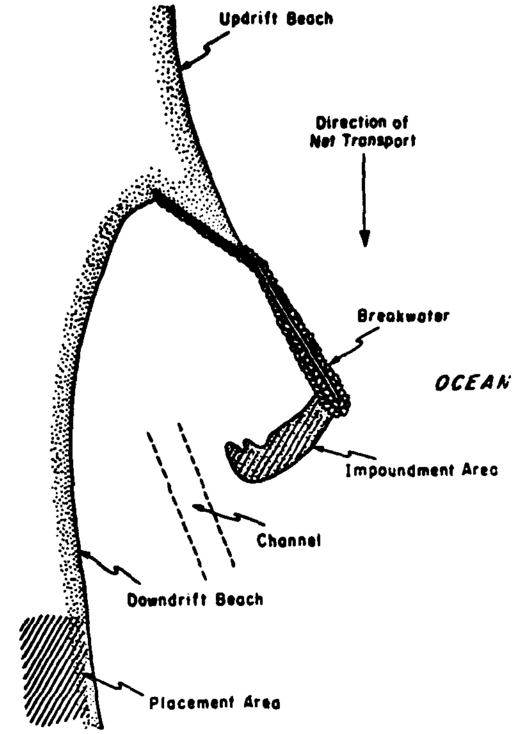

Figure 2. Impoundment area at distal end of a shore-connected breakwater.

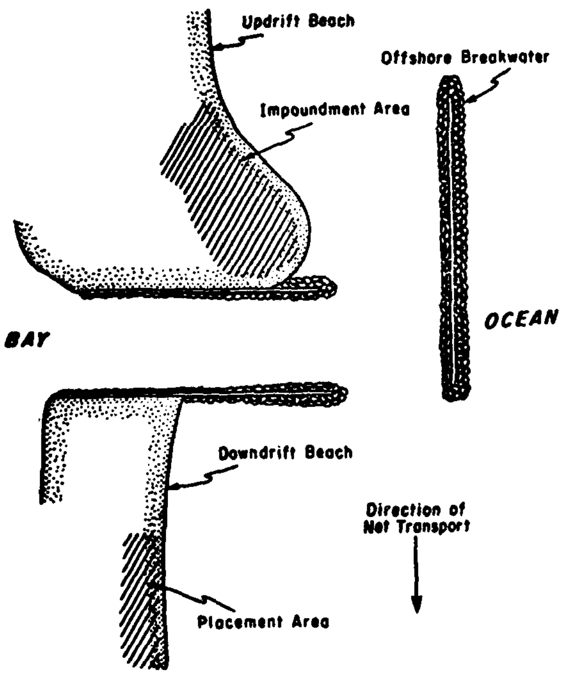

Figure 3. Impoundment area behind detached offshore breakwater.

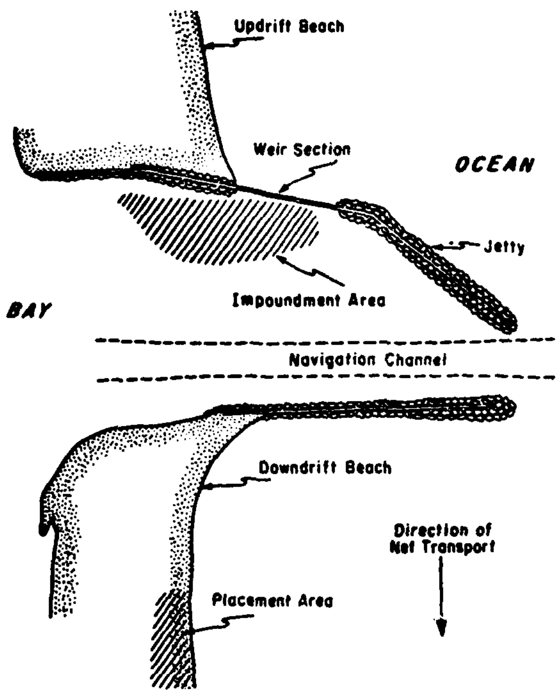

Figure 4. Weir jetty with impoundment area between jetties.

NOTE:--Figures 1 to 4 modified from Watts (1965).

area is protected from waves by the weir and jetty. Sand is pumped from the deposition basin to nourish the downdrift beach. The jetties fix the location of the navigation channel; the updrift jetty controls the transport of sand into the deposition basin, controls alinement of the updrift beach, and provides protection to a dredge operating in the deposition basin.

II. WEIR-JETTY SYSTEM

The key elements of a weir-jetty system (Fig. 5) are: (a) An updrift jetty comprised of a sandtight landward section, a weir section with an elevation near mean sea level (MSL), and a seaward section with a typical jetty cross section; (b) a downdrift jetty which normally has a typical jetty cross section without a weir; (c) a deposition basin; (d) a navigation channel; (e) an updrift beach; and (f) a downdrift beach which normally also serves as the disposal area for sand removed from the deposition basin. The design objectives of each element are discussed below and, in more detail, in Section III.

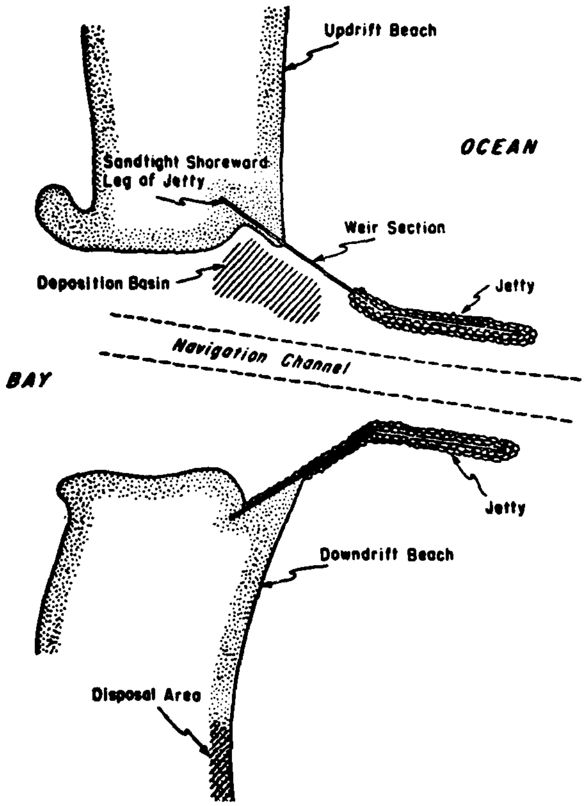

Figure 5. Key elements of a typical weir-jetty system.

A weir-jetty system is a multiple-purpose coastal structure system that serves what at times may be conflicting uses. It primarily serves navigation by keeping the location of the channel through a tidal inlet relatively fixed with adequate water depths to provide safe passage for vessels. The remaining functions arise because of a need to bypass sand to downdrift beaches and to improve flushing of the navigation channel.

The general design objectives of any inlet stabilization or harbor project are to provide a safe navigation channel with adequate dimensions, to minimize the need for channel maintenance dredging, and to preclude or minimize any adverse effects of the project such as downdrift beach erosion. The mitigation of downdrift erosion is a purpose of bypassing sand as is keeping the navigation channel free of sand. A weir-jetty system will meet these objectives if (a) the navigation channel is kept in a fixed location and relatively free of sediments, (b) the weir section and outer jetty section of the updrift jetty provide wave protection to a dredge in the deposition basin, and (c) the overall jetty complex provides wave protection to vessels using the channel.

Another hydraulic function of a weir-jetty system is to allow flood currents to enter the inlet over the weir during floodflow with subsequent channeling of ebb flows out of the inlet between the jetties. During ebb flow most of the tidal prism should exit the inlet through the navigation channel (between the jetties) with a minimum of flow exiting across the weir. This ebb-flow dominance results in higher ebb tidal currents in the navigation channel and tends to flush sediments from the channel. These sediments may be deposited in an outer bar, indicating a need for longer jetties. In addition, the weir and jetty provide wave protection to vessels navigating the inlet and provide protection to a dredge bypassing sand from the deposition basin to the downdrift beach.

The weir jetty also serves as a structure for controlling sediment transport into the inlet by providing a low sill over which sand is transported by waves and thus determining the location within the inlet where deposition occurs. By limiting deposition to a predetermined area, sand is kept out of the navigation channel and deposited where a dredge can safely operate.

The ideal weir-jetty system will minimize the amount of sand which needs to be bypassed. Optimally, this minimum is the net sand transport into the inlet. If $q(t)$ is the rate of longshore sand transport at a point on the beach at a time, t, then the transport of sand as a function of time at that point can conceptually be described as shown in Figure 6. The sign convention of the figure assumes that q is positive when transport is toward the right of an observer looking seaward from the beach; when q is negative, transport is toward the left. The net transport *rate* is simply the average value of $q(t)$ given by

$$q_{net} = \frac{1}{\tau} \int_{t_0}^{t_0+\tau} q(t)\, dt \tag{1}$$

where τ is usually chosen to be 1 year. The total net transport during the time interval, τ, is

$$Q_{net} = q_{net}\, \tau = \int_{t_0}^{t_0+\tau} q(t)\, dt \tag{2}$$

The total transport to the right during τ is

$$Q_R = \int_{t_0}^{t_1} q(t)\, dt + \int_{t_2}^{t_3} q(t)\, dt + \ldots + \int_{t_n}^{\tau-t_n} q(t)\, dt \tag{3}$$

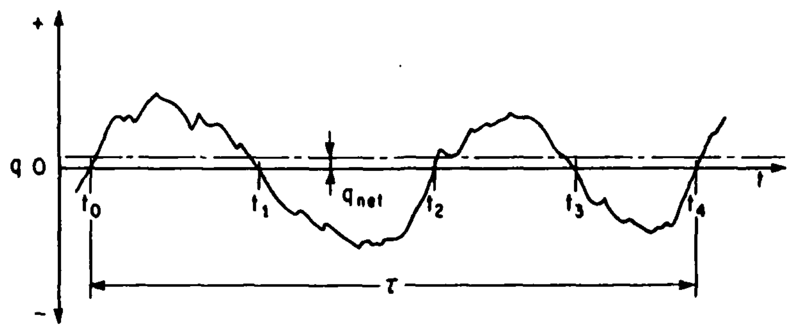

Figure 6. Time history of longshore sand transport past a point on the shoreline.

where the integrals are evaluated only for those time periods when $q(t)$ is positive; the total transport to the left during τ is

$$Q_L = -\int_{t_1}^{t_2} q(t)\,dt - \int_{t_3}^{t_4} q(t)\,dt - \ldots - \int_{t_n}^{\tau} q(t)\,dt \qquad (4)$$

where the integrals are evaluated for only those time periods when $q(t)$ is negative. The minus sign is included so that Q_L will be a positive number. Therefore, the net volume transport is

$$Q_{net} = Q_R - Q_L \qquad (5)$$

The gross transport, defined as the total amount of sand passing the point on the beach during τ is

$$Q_{gross} = Q_R + Q_L \qquad (6)$$

Sand transport at an ideal weir-jetty system is shown in Figure 7. Assuming that the net transport in the figure is from left to right (i.e., $Q_R > Q_L$), the minimum amount of sand that should need to be bypassed for erosion control is Q_{net}. In an optimum system, only Q_{net} would enter the deposition basin for bypassing to the downdrift beach. The amount of sand carried to the weir from the updrift beach is Q_R, which will be larger than Q_{net} if a net transport is to the right as assumed. An amount of sand equal to $Q_R - Q_{net}$ must therefore be retained in temporary storage on the updrift beach to keep the sand from moving into the deposition basin. The sand on the updrift beach will then be available to replace the material trapped by the downdrift jetty and be transported back up the beach during periods of transport reversal; i.e., when the transport is to the left. The quantity of sand held in temporary storage is $Q_R - Q_{net}$ which, from equation (5), is equal to Q_L. Not all of this sand needs to be held in storage at one time. The amount of storage required in the updrift beach will depend on the frequency and magnitude of reversals in transport. A suggested analysis procedure to determine the amount of storage needed is presented in Section VII.

The bypassing requirements described above are for an ideal weir-jetty system. However, several factors preclude achieving this optimum situation.

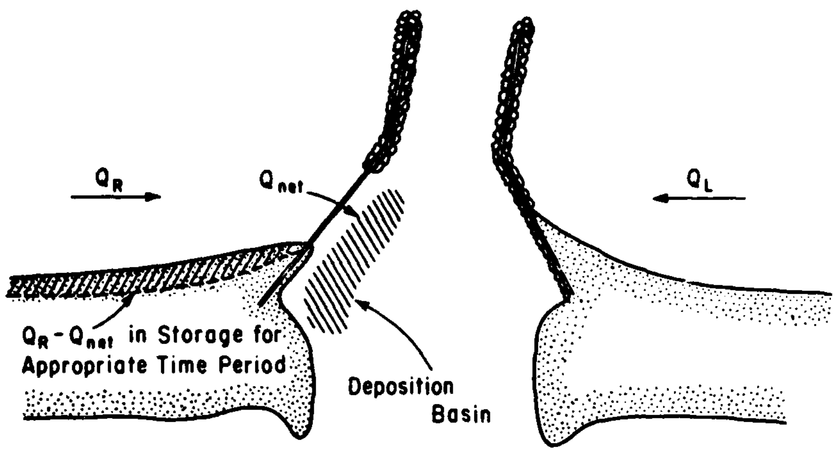

Figure 7. Sand transport and storage at an ideal weir-jetty system.

The longshore transport environment does not remain the same from year to year. In some years the net transport may be in one direction; in other years it may be in the opposite direction. If the period of record used to determine transport quantities and directions is atypical, serious errors in predicting the performance of a weir system can result. Another factor which complicates predicting bypassing quantities is that transport conditions at a given time may differ from one side of the inlet to the other because of wave refraction caused by complex bathymetry near inlets and differences in shoreline alinement. Although the optimum bypassing situation which requires minimum material handling may not be achieved, it is a goal toward which the design should be aimed. In addition to controlling the amount of sand entering the deposition basin, the sandtight landward leg of the weir jetty and the overall jetty layout act to control the planform of both the updrift and downdrift beaches. The location of the landward end of the weir determines how wide the updrift beach will be, how much sand stored in the updrift beach is available for transport back up the beach during reversals in wave direction, and how much sand will find its way into the deposition basin.

The various functions of the weir dictate different system characteristics which sometimes conflict with each other. For example, to maximize wave protection for a dredge operating in the deposition basin, the weir-crest elevation should be as high as practical; however, to achieve the desired control of sedimentation, a lower weir crest is needed. A lower weir crest is also desirable for maximizing the amount of flow entering the inlet during floodtide. A higher weir crest would be desirable during ebbtide to contain the flow between the jetties. These conflicting functions require trade-offs to achieve an optimum overall system. They suggest that flexibility be engineered into any weir-jetty design so that adjustments can be made after construction when project performance has been observed.

At the present state-of-the-art and with the limited information usually available on longshore transport at a site, an optimum weir-jetty design may not be attainable. Additionally, the transition from the equilibrium that exists before a weir-jetty system is constructed to a new postproject equilibrium requires some as yet undetermined time to be attained. During this transition, performance of the system may not truly reflect its capability, and adjustments based on observations during this time may ultimately prove

ill-advised. A designer must keep the ideal weir system as a goal but may need to compensate for the consequences of uncertainties in understanding the processes and data. In some instances designers may need to compromise on bypassing capabilities and design for quantities greater than the net transport.

III. ELEMENTS OF A WEIR-JETTY SYSTEM

The design of a weir-jetty system requires that at least six elements of the system be considered: (a) The navigation channel, (b) the jetty structures, (c) the weir structures, (d) the deposition basin, (e) the updrift beach, and (f) the downdrift beach. The design of each of these elements is governed by the hydraulic characteristics of the inlet tides and tidal range, wave and longshore transport climate at the site, the size and type of vessels using the inlet, and the overall inlet geometry. Design factors for each element of the weir-jetty system are discussed below.

1. Navigation Channel.

Since the primary purpose of a jetty system is to maintain a fixed navigation channel, improvement of navigation conditions at an inlet must be the prime consideration in any weir-jetty design. The depth, width, and alinement of the channel are parameters that need to be established and are usually a compromise between what is needed to serve navigation and what the physical conditions at the site will allow. Channel depth and width are determined by the size, type, and number of vessels which are using or will eventually use the inlet. Constraints are imposed on the depth and width by inlet hydraulics. For example, the tidal prism may not be large enough to keep the proposed navigation channel open with realistic maintenance dredging efforts and a smaller channel cross section must then be considered. Inlet hydraulics and cross-section stability are discussed by Sorensen (1977). Channel alinement is dictated by navigation requirements as well as local inlet geometry and sedimentation processes. Existing shoals usually establish the most economic jetty alinement and thereby influence the location of the channel. Navigation needs, such as required turning radii and maneuvering areas for safe navigation, also influence channel alinement. The approach direction of prevailing waves is another factor in establishing channel and jetty alinement. If small craft are to be protected from wave action in the channel, the jetties and entrance channel should be alined to afford maximum wave protection. Wave steepening at the seaward end of the jetties caused by opposing ebb tidal currents may also be a critical factor in achieving safe navigation conditions. The steepened waves may break over the ocean bar making navigation dangerous.

2. Jetty Structures.

Design factors which must be determined for jetty structures are alinement, spacing, and structural considerations such as construction type, crest elevation, and structural design.

Jetty alinement is mostly governed by the geometry of the navigation channel, inlet, and shoals. Designs are usually selected which minimize the overall cost of the structures by making maximum use of shallow water over existing shoals. The cost of rubble structures rises rapidly as the water depth in which they are built increases. Historical data of inlet migration and shoaling patterns should be reviewed to provide information on future tendencies of the navigation channel to migrate. The jetties should be

located to take advantage of any beneficial natural processes, provided those processes will continue after project construction. Navigation requirements may also influence alinement. In addition to alining the jetties to provide maximum protection for vessels navigating the inlet, the characteristics and maneuverability of vessels using the inlet may be a factor.

The distance between jetties is governed by both navigation and hydraulic factors. As with the design of the navigation channel itself, the size of the tidal prism, for the most part, determines the cross-sectional area of the inlet throat and of the channel between jetties. Jetty spacing, therefore, has some influence on the relative dimensions of the channel, i.e., the channel width-to-depth ratio. Jetties spaced too far apart will encourage shoaling with inadequate water depths to serve the vessels for which the channel is intended; jetties too close together may result in channel scouring and cause scour holes which can undermine jetties and eventually require considerable efforts to prevent a major structural failure. Closely spaced jetties may also endanger vessels by increasing the possibility of collision with other vessels or the jetties and by causing excessive wave steepening over the ocean bar.

The primary factor influencing the structural design of jetties is the local wave and water level climate. Structural details include establishing the crest elevation, structure type, and the structure cross section. For economic as well as technical reasons, rubble-mound construction is preferred for jetties because the low maintenance required usually results in minimal annual cost, even though other types of construction such as steel or concrete sheet-pile jetties may have a lower initial cost. Rubble structures are considered "flexible" structures. When subjected to waves exceeding their design level, the damage they experience is usually progressive, making repairs relatively simple and inexpensive. Rubble structures also continue to provide protection when in a damaged state. Destruction of less flexible structures can be catastrophic with complete loss of function.

Sheet-pile and caisson-type jetties reflect wave energy and encourage the formation of seaward-moving currents adjacent to the structures. For weir jetties where currents could transport sand away from the weir section into deeper water and eventually into the navigation channel, rubble-mound construction is preferred. Wave reflection from the structure is also minimized. The prediction of updrift and downdrift beach planforms is simpler if wave conditions adjacent to the structure are not confused by reflected waves. Rubble jetties also decrease possible adverse effects of reflected waves on navigation conditions. Jetty crest elevation is usually selected to prevent overtopping for some design wave and water level condition. It is generally not possible for a design to preclude overtopping since designing for extreme waves and water levels is usually not economically justified. The effects of exceeding design conditions for wave and water levels should, however, be investigated and the design optimized by balancing the higher initial cost of a more substantial structure against the higher maintenance and repair costs and decreased benefits of a less substantial structure. Some savings may accrue by decreasing armor-stone size near the landward end of the jetties since wave heights are usually depth-limited. Armor near the shore will be subjected to smaller waves or may even be insulated from wave action by the sand that accumulates against it. However, some estimate of scour is necessary in order to not underestimate the local wave height which could result in underdesigned armor units.

Unless economical and practical factors dictate otherwise, rubble-mound structures should be used for jetty construction.

3. Weir Structure.

Factors required in designing the weir section of a jetty include determining weir length, orientation, elevation, construction type, and location of the landward end of the weir section. The weir length is selected so that it extends through the normal surf zone and thus intercepts most of the sand in transport along the beach. Laboratory experiments and qualitative field observations indicate that much of the sand transported over a weir structure crosses near the beach face and that the beach profile adjacent and updrift of the weir adjusts and flattens to allow significant bedload transport over the weir in the region where the beach, weir, and waterline intersect. The location of maximum transport on the beach face changes with tidal stage. In the breaker region adjacent to the weir, suspended sediments are also carried over the structure. The amount of transport over the weir in this region is probably sensitive to the weir elevation, tidal stage, and level of wave activity. To intercept this transport under most conditions, the weir section should extend beyond the normal breaker location. Protection for a dredge from wave action in the deposition basin is also a factor in establishing weir length. The weir section allows some waves to overtop it and at times may allow excessive wave action in the deposition area which would suspend bypassing operations. To minimize this downtime, the weir section should be as short as possible. Weir elevation also influences the level of wave activity in the deposition basin. The length of the weir section on existing weir jetties varies from about 580 to 1,800 feet (see Table 1 which presents the characteristics of the weir sections and wave conditions at six existing weir jetties in the United States). The length of these weirs reflects designer concern for "sanding-in" of the weir section. Sanding-in may occur during storms when large quantities of sand reach the jetty but are not efficiently transported over the weir into the deposition basin. Observations of the performance of existing weir jetties suggest that this may not be as serious a problem as first believed. In any event, the design of weirs in areas where large quantities of sediment may be transported during short time periods should consider the possibility of sanding-in.

Weir elevation is established with sediment transport, wave attenuation, tidal range, and tidal current considerations in mind. Sediment transport considerations dictate that the weir be as low as possible. In fact, for transport purposes, the weir could possibly be omitted altogether and only the bottom armored to fix the beach at a desired profile. The weir would thus serve as a template for the updrift beach. The necessity of wave protection for a dredge operating in the deposition basin behind the weir, however, requires that the weir elevation be as high as possible. A compromise weir elevation must be established that will functionally serve both sediment transport control and wave protection. The elevation of existing weirs is given in Table 1. Generally, the weir elevation has been set at the mean tide level (MTL) in areas where the tidal range is about 2 to 5 feet (Atlantic coast) and at mean low water (MLW) in areas with a relatively low tidal range (gulf coast). This appears to be a satisfactory compromise, but one that should be investigated in the design of any weir-jetty system. In regions with a large tidal range (12 to 15 feet) weir jetties are generally not a viable alternative since the transport of sediment over the weir is limited to only a small part of the tidal period.

Table 1. Characteristics of weir sections and wave conditions at existing weir-jetty complexes.

Location	Elev. (MLW)	Lgth. (ft)	Orien.[1]	Cross section	Tidal range (ft)	Wave gage data: Avg. annual significant wave hgt. (ft)	Wave gage data: Wave period (s)	Dominant wave direction[2]	Gross rate of longshore transport[3] (yd^3)	Predominant rate of longshore transport[3] (yd^3 and direction)	Trap-filling rate[3] (measured yd^3/yr)
Hillsboro Inlet, Fla.	+0.5 to +3.5	580	0°	Gently sloping natural rock	Mean range 2.3 Spring range 2.7	Lake Worth: Feb. 1966 $H_s = 2.40$ $\sigma = 1.67$	July 1967 $\bar{T} = 5.82$ $\sigma = 2.56$	ENE.	-------[4]	Southward	60,000 to 100,000
Masonboro Inlet, N.C.	+2.0	1,000	85°	Vertical concrete sheet pile	4.0 4.7	Wrightsville: Apr. 1971 $H_s = 2.55$ $\sigma = 1.10$	Oct. 1974 $\bar{T} = 7.79$ $\sigma = 2.42$	ENE.	340,000	220,000 Southward	143,000
East Pass, Fla.	-0.5	1,000	45°	Vertical concrete sheet pile[5]	0.6 Diurnal	Destin: Sept. 1971 $H_s = 1.85$ $\sigma = 1.11$	June 1974 $\bar{T} = 5.79$ $\sigma = 1.75$	SE.	195,000	130,000 Westward	47,000 to 95,000
Perdido Pass, Ala.	-0.5	1,000	45°	Vertical concrete sheet pile[5]	0.6 Diurnal	Destin: Sept. 1971 $H_s = 1.85$ $\sigma = 1.11$	June 1974 $\bar{T} = 5.79$ $\sigma = 1.75$	SE.	195,000	130,000 Westward	200,000
Ponce de Leon, Fla.[6]	+4.0 0	300 1,500 1,800	60°	Vertical concrete king piles and panels[7]	2.3 2.7	Daytona Beach: Nov. 1964 $H_s = 1.92$ $\sigma = 1.02$	May 1968 $\bar{T} = 8.85$ $\sigma = 2.11$	ENE.	700,000	600,000 Southward	300,000
Murrells Inlet, S.C.	+2.2	1,350	30°	Rubble	4.4 3.3	Myrtle Beach: Jan. 1975 $H_s = 1.93$ $\sigma = 0.88$	Apr. 1977 $\bar{T} = 6.89$ $\sigma = 2.16$	ENE.	250,000	150,000 Southward	-------[4]

[1]Angle from general trend of shoreline measured on the channel side.

[2]From Shore Protection Manual (SPM) (U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1977).

[3]Annual rate; gross and predominant rate data from Jarrett (1976).

[4]No data available.

[5]After structural failure at East Pass, the weirs at East and Perdido Passes were armored with rubble-mound aprons.

[6]Length of weir section at Ponce de Leon was shortened in 1978 by placing 500 feet of rubble over landwardmost end of weir section.

[7]Armored with rubble-mound section during construction because of scour and breakage of panels by vibration.

The alinement of the weir section, with respect to the updrift beach, is usually dictated by inlet geometry and the presence of shoals that can form a satisfactory foundation for the structure. Construction over existing shoals will lower costs by minimizing the amount of materials needed and by alining the structure to achieve the shortest possible jetty length. The effect on sediment transport characteristics of weir-section alinement, with respect to the shoreline, does not appear to be significant. Existing weir jetties have weir alinements ranging from shore-parallel weirs to weirs that are perpendicular to the updrift shoreline (see Table 1). The weir at Hillsboro Inlet, Florida, which served as the prototype for the weir-jetty concept, is nearly parallel with the updrift shoreline; the weir section in the north jetty at Masonboro Inlet, North Carolina, is nearly perpendicular to nearby Wrightsville Beach. The weir sections at both Murrells and Little River Inlets, South Carolina, are alined at an angle of approximately 45° with the updrift beach. Although this range of angles is relatively large, the sediment transport conditions at any of the existing weirs do not appear to be impaired. Shore-parallel weirs are preferable since the possibility of sand being transported past the sandtight seaward end of the jetty is smaller. Thus, it is more likely that all of the sand transported to the weir will be carried over it into the deposition basin with less sand eventually entering the navigation channel.

The size and location of the deposition basin influence the weir alinement which must provide adequate area for locating a basin within the inlet. The basin must also be positioned to trap the sand transported over the weir; ideally, it should be placed to afford maximum protection for a dredge operating in the basin. These factors vary from site to site and depend on prevailing inlet geometry.

A critical jetty design factor is to establish the location of the landward end of the weir section. The jetty section connecting the weir with the shoreline should be sandtight to hold the updrift beach in a dynamically stable planform. The length of the sandtight shore section is determined from the desired updrift beach configuration. If the sandtight section is too short, the erosion which occurs during reversals may endanger a significant area updrift of the inlet; if too long, a large volume of sand accumulates in the updrift beach. Ideally, the amount of sand stored in the updrift fillet should be the amount needed to nourish updrift beaches when the longshore sand transport is in the updrift direction.

Weir sections in earlier projects (Perdido Pass, Alabama; East Pass, Florida; and Masonboro Inlet) are of sheet-pile construction; recent projects (Murrells and Little River Inlets) have been designed with rubble-mound weirs. Sheet-pile weir sections were constructed because of their relatively low initial cost when compared to rubble construction; however, smooth-faced sheet-pile weirs have been found to reflect waves and cause confused wave conditions in their vicinity. The effect of these confused wave conditions on sediment transport across the weir is unknown; however, they are considered detrimental. Currents along the weir are also more likely to occur if sheet-pile construction is used. These currents may result in a decrease in transport over the weir and an increase in transport along the weir, with the sand eventually entering the navigation channel. Present practice is to construct the weir section of rubble with one or more rows of quarrystone with their crests at the desired weir elevation to minimize the confused wave conditions

conditions at the weir and also to decrease the tendency for adverse currents to form. The amount of wave action on the leeward side of the weir is also reduced because of the better wave attenuation characteristics of a rubble weir. Figure 8 shows a typical rubble-weir section.

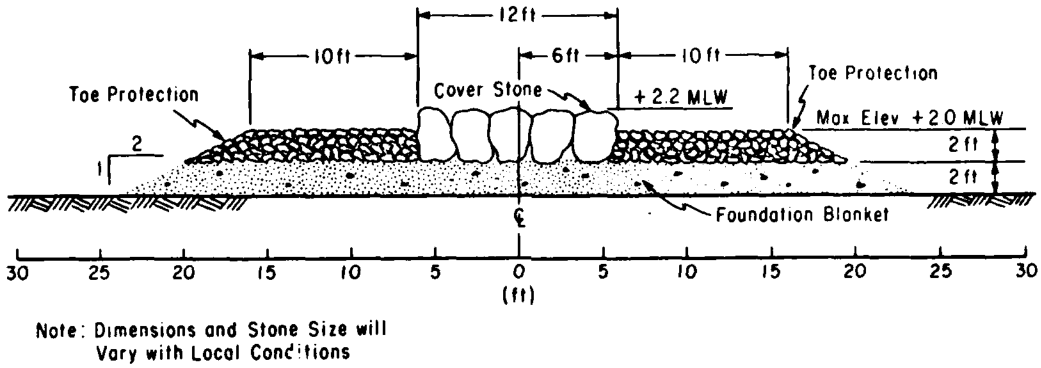

Figure 8. Typical weir cross section for a rubble-mound weir.

Rubble structures provide some flexibility in adjusting the weir to accommodate unforseen transport condition variations for which the system may not have been designed. Since the current ability to accurately and adequately describe the longshore transport environment is poor, flexibility in adjusting the transport characteristics of the weir section is highly desirable to cope with reversals and anomalies in the period of record. Early attempts to achieve flexibility by use of king piles with removable panels were unsuccessful, mainly because of structural problems with removing and inserting panels after the structure settled in response to wave and soil forces. These problems do not arise with rubble construction although any modification of a rubble-weir section is costly and requires the use of heavy construction equipment. Cost was a factor in selecting rubble weirs for both Murrells and Little River Inlets where estimated construction costs for sheet-pile weirs exceeded the cost of the selected rubble-weir configuration. The use of rubble for the weir section is strongly recommended.

The performance of the north jetty at St. Lucie Inlet, Florida, suggests the possibility of designing a permeable jetty that functions as a weir. An advantage of such a system is the increased wave protection afforded to a dredge operating in its lee. St. Lucie Inlet, however, is not typical since the deposition area is not in the lee of the updrift jetty but in a spit that develops inside the inlet (Fig. 9). Sediment passing through the weir is carried by flood currents into the inlet where it is deposited in a spit which grows along the inlet shore into the bay. The spit is dredged periodically and the sand bypassed. The effectiveness of the system is partly attributable to the local inlet geometry and to the particular wave conditions at the site. However, a permeable weir may be more costly because of the additional stone required to fill the weir section.

The choice of weir type and alinement largely depends on the (a) site-specific conditions, (b) location of existing shoals, (c) availability of a suitable deposition basin area either adjacent to the weir or farther back in the inlet, and (d) relative cost of the various alternative types of construction.

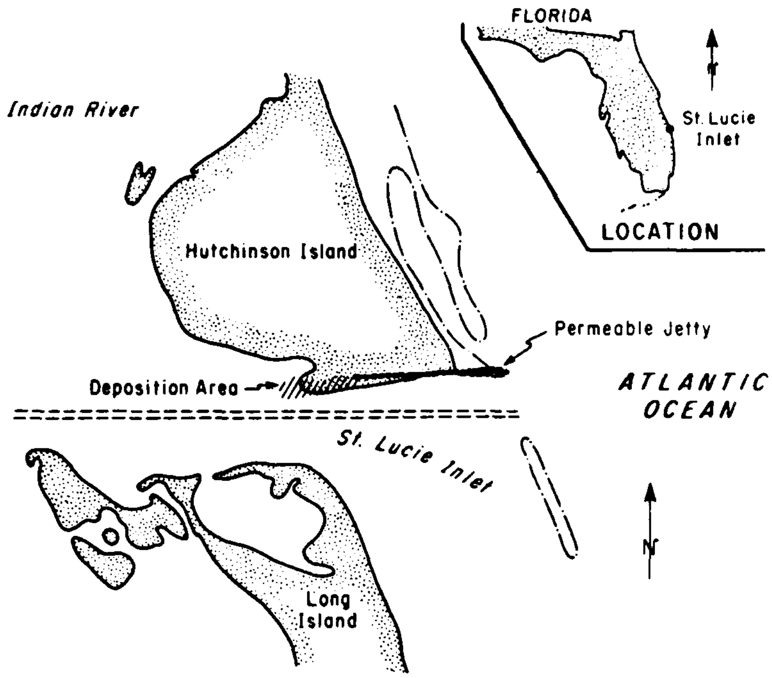

Figure 9. Deposition area in a spit at St. Lucie Inlet, Florida.

4. Deposition Basin.

Characteristics that must be determined in a design of the deposition area are basin location, shape, and capacity.

Basin location and shape are dictated mostly by existing inlet geometry and the desired location of the navigation channel. The deposition area is usually adjacent to or behind the weir section as in the case of a shore-parallel weir (e.g., at Hillsboro Inlet). The basin should be positioned far enough from the base of the weir and jetty to preclude the jetty from sliding into the basin as a result of slope failure. In some cases, the deposition area may be located a distance from the weir in the form of a sandspit that may develop in the bay behind the inlet throat, e.g., St. Lucie Inlet. The expected response of the navigation channel to jetty construction must be considered in selecting the location of the deposition basin. Sheltering of the navigation channel by the jetties and thus excluding the normal sand transport into the channel usually results in channel reorientation shortly after construction. Removal of some factors that hold the unimproved inlet in a state of equilibrium may lead to channel migration into the proposed deposition area. Providing room for a deposition basin between two jetties usually requires an "arrowhead" jetty layout or a modification thereof. If there is a tendency for the navigation channel to meander, its movement into the deposition basin is possible. In this case, a training dike may be required to fix the channel location between the jetties in the reach from the inlet entrance at the seaward end of the jetties, past the deposition basin and into the inlet throat (Fig. 10). Construction of such a dike, if needed, will contribute to the cost of a weir-jetty bypassing system even though its crest elevation need not be high.

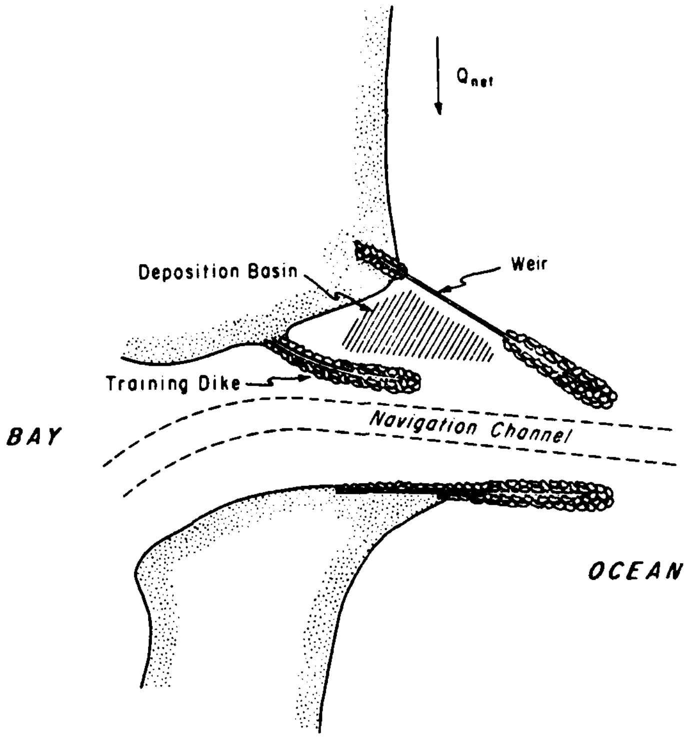

Figure 10. Training dike to control location of navigation channel in a weir-jetty system.

Two factors which influence the required deposition basin capacity are the longshore sand transport rates into the inlet and the estimated frequency at which the basin will be dredged. At sites where physical constraints on deposition basin size may exist due to inlet geometry, the longshore transport rate and maximum allowable deposition basin size will establish the frequency of dredging.

Existing systems are intended to be dredged annually but the deposition basins have been designed to hold a 2-year storage as a safety factor. Thus, the required capacity of the basin will be twice the annual volume transport into it. The amount transported into the basin will be between a minimum equal to the net transport at the inlet and a maximum equal to the total downcoast transport. That is, if the annual downcoast transport is Q_R and the upcoast transport is Q_L with $Q_R > Q_L$, then the amount trapped per year will be between the minimum of $Q_R - Q_L = Q_{net}$ and a maximum of Q_R, assuming that no sand is lost offshore. If dredging is scheduled for a 2-year cycle, the capacity of the basin should be $2Q_R$ to assure adequate storage volume. If the annual transport is highly variable from year to year, additional deposition basin capacity may be necessary to provide sufficient storage for 2 consecutive high-transport years. An alternative is to dredge more frequently during such anomalous occurrences.

In the first few years after jetty construction the amount of sand entering the deposition basin may exceed the normal longshore transport to the

inlet. Because of the construction, ebb tidal currents will no longer hold part of the ocean bar offshore of the inlet; the ebb currents are directed offshore in a narrow hydraulic jet allowing waves to move a part of the offshore bar shoreward and eventually into the deposition basin. Dredging during the first several years after construction may therefore exceed normal requirements. A part of the offshore bar may remain seaward of the newly constructed jetties but it will be much smaller in size than the preconstruction bar.

Initial dredging of the deposition basin and navigation channels often requires removal of more sand than is necessary for construction of sand dikes or other project appurtenances. Although the material may not be required initially, it is a valuable resource and should be stockpiled as dunes along the updrift and downdrift fillets for possible future use as beach nourishment along adjacent beaches and as an additional defense against breaches outside the jetties.

5. Updrift Beach.

Alinement of the updrift beach is governed by the wave environment and wave reflection, refraction, and diffraction in the vicinity of the jetty structures. Generally, beach width near the updrift jetty is established by the length of the sandtight, shore-connected leg of the jetty. Shoreline location along the updrift beach at any instant depends on the longshore transport history at the site. After extended periods of downcoast transport, the fillet adjacent to the weir will usually be full and the beach wide; after periods of upcoast transport the fillet may be empty (except for the sheltered area immediately in the lee of the jetty) and the beach relatively narrow. The magnitude of fluctuations in shoreline location is determined by wave height, period, and direction variability. The condition of the updrift beach at any time depends on the transport conditions that prevailed before the time of observation. Prediction of beach response requires a knowledge of the longshore transport environment that includes the frequency and duration of reversals as well as the net and gross transport rates. Several methods which are available for predicting beach response to the construction of jetties range from simple extrapolation of the existing updrift beach alinement toward the inlet to the mathematical simulation of beach changes by performing sediment balance calculations for small beach cells.

6. Downdrift Beach.

Shoreline alinement, range of shoreline fluctuations, and location of the bypassed sand disposal area are characteristics of the downdrift beach that need to be determined. Like the updrift beach, the alinement of the downdrift beach and the range of onshore-offshore movement of the shoreline depend on the longshore transport environment and transport history at the site, the frequency and magnitude of bypassing operations, and where the bypassed sand is placed along the downdrift beach. If the bypassed sand is placed too close to the downdrift jetty, the sand could move into the lee of the jetty and fail to nourish downdrift beaches.

Timing of the bypassing operation is also a factor. If bypassing is performed when transport is in the upcoast direction, bypassed sand will be carried toward the inlet and may enter the navigation channel. Bypassing

should be scheduled for times during the year when there is a high probability that bypassed sand will be carried away from the inlet to downdrift beaches. Consideration of all these factors requires knowledge of the transport environment, particularly seasonal variations in transport direction and magnitude.

Planning should take into account acquisition of easements along areas of both the updrift and downdrift beaches for placement of bypassed sand and for stockpiling sand. The areas, which serve as feeder beaches to provide sand to adjacent areas, must be located far enough from the inlet to avoid the wave shadow of the jetty structures and to preclude large amounts of sand from returning to the inlet during short-term reversals in transport.

The same methods for predicting updrift beach response to the construction of jetties can be applied to the downdrift beach (discussed in more detail in Section X).

IV. WEIR HYDRAULICS

Weir jetties serve a hydraulic function. During floodtide when water levels exceed the weir-crest elevation a part of the inlet's tidal prism flows across the weir into the inlet. During ebb flow, much of the water that has entered the inlet over the weir flows out through the navigation channel. The weir thus causes a greater ebb flow out between the jetties than enters between the jetties during floodflow.

Greater ebb flow causes ebb current velocities in the navigation channel to exceed flood current velocities and results in natural flushing of sediments from the channel. The amount of ebb dominance resulting from the weir depends on several factors. The phase lag of the tide level on the weir's channel side behind the tide level on the oceanside causes a head difference and drives a current across the weir. In addition, the relative amplitude of the tide on each side of the weir influences the current. Typical tidal curves measured across a laboratory weir jetty are shown in Figure 11. Phase lag and tidal amplitude difference across the weir depend on the inlet hydraulics as characterized by Keulegan's K (see Sorensen, 1977) and by the proposed jetty system geometry.

A major factor influencing velocity assymetry is weir-crest elevation. Lower weirs allow more flow to enter the inlet during floodtide, but also allow more ebb flow to escape across the weir. Waves also transport water over the weir and contribute to ebb-flow dominance. Since higher waves generally act only on the weir's oceanside, wave transport of water across the weir is into the inlet. There is little or no corresponding seaward wave transport out of the inlet. Wave setup on the oceanside of the weir also contributes to the head difference across the weir and causes flow into the inlet. Each of these factors contributes toward keeping ebb current velocities greater than flood current velocities through the navigation channel and thus assists in preventing channel shoaling.

The amount of water carried over the weir because of tidal phase and amplitude differences between the ocean and channel sides of the weir can be estimated using an appropriate weir flow formula. If the weir section has a well-defined crest elevation such as would exist for a sheet-pile weir, the discharge per unit length of weir crest can be calculated from (see Fig. 12)

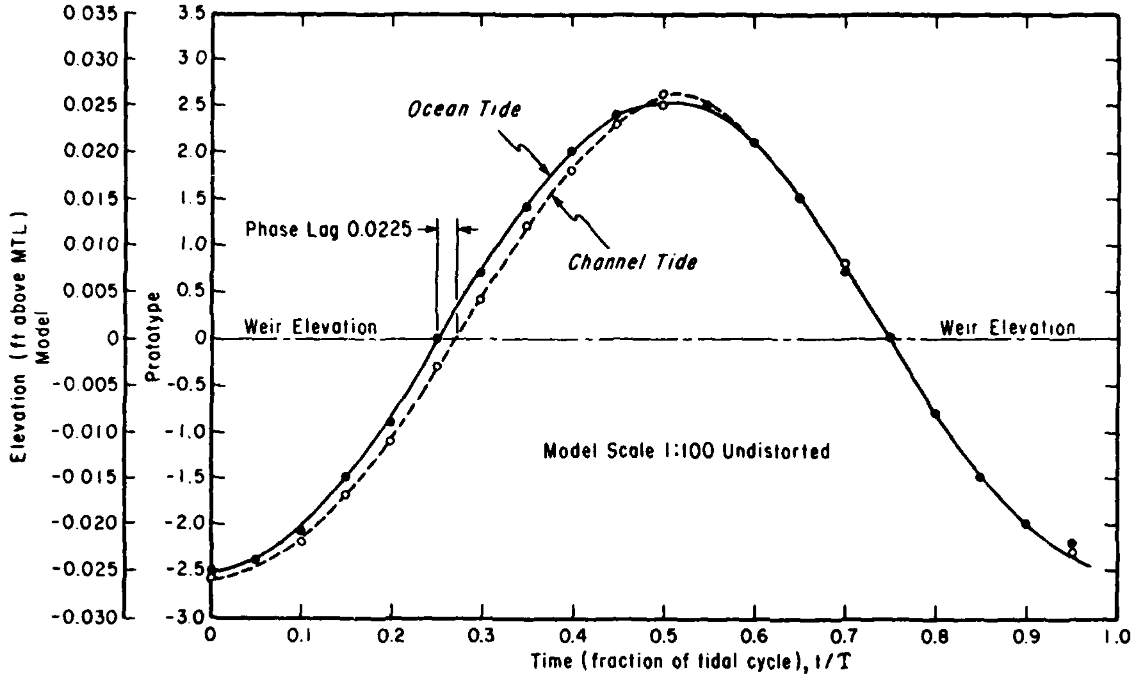

Figure 11. Phase lag between ocean tide and channel tide across a model weir jetty (weir at midtide level).

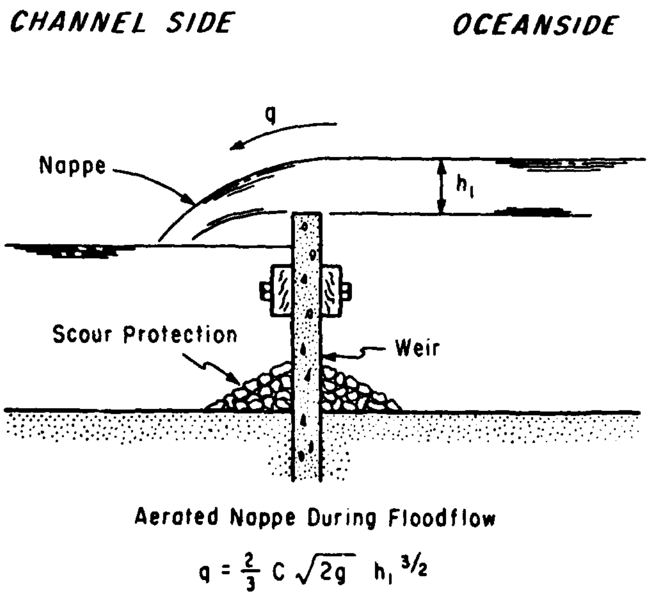

Figure 12. Definition sketch of flow across weir with aerated nappe.

$$q = \frac{2}{3} C \sqrt{2g} \, h_1^{3/2} \qquad (7)$$

where

- q = discharge of water per unit of weir-crest length
- C = weir discharge coefficient (which for the required accuracy can be assumed constant = 0.6)
- g = acceleration of gravity
- h_1 = head above the weir crest

Equation (7) applies when the nappe of the overflow is aerated, i.e., when the water level on the downstream side of the weir is below the weir-crest elevation. This situation occurs for only a short time during a tidal cycle since the phase difference between the tides on each side of the weir is usually small (Fig. 13). The more frequent situation occurs when the weir crest is submerged as it is during most of a tidal cycle. For this case, the discharge is given by

$$q = C \sqrt{2g(h_1 - h_2)} \left(\frac{2}{3} h_1 + \frac{1}{3} h_2 \right) \qquad (8)$$

where h_2 is the downstream head over the weir crest and the other variables are as defined for equation (7) (Fig. 14). The calculation of weir discharge is illustrated by example problem 1.

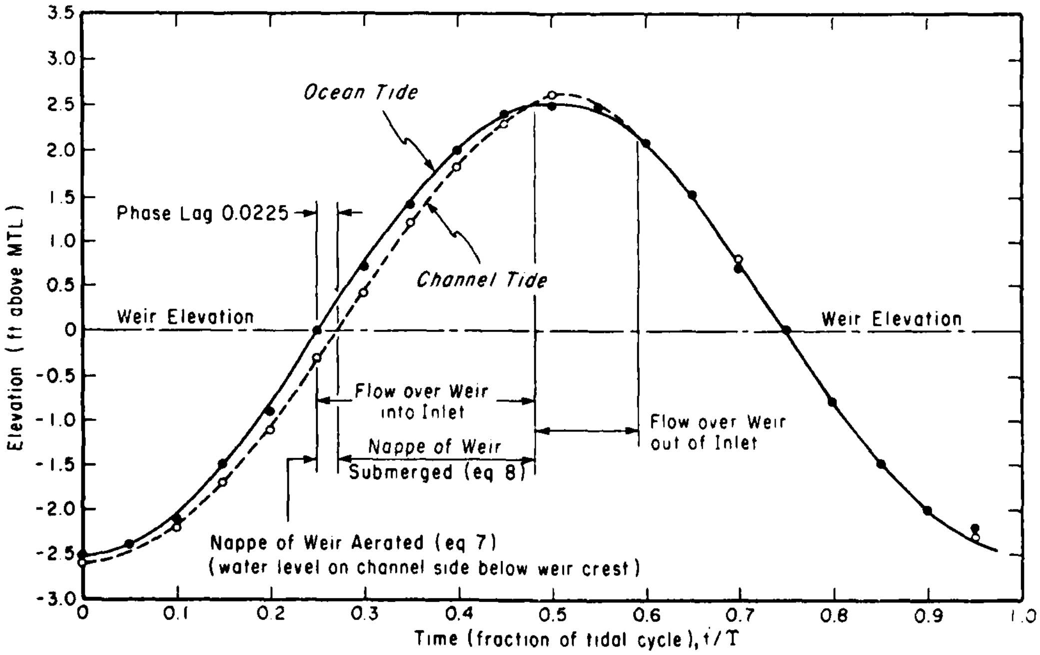

Figure 13. Conditions of weir flow at various times during a tidal cycle.

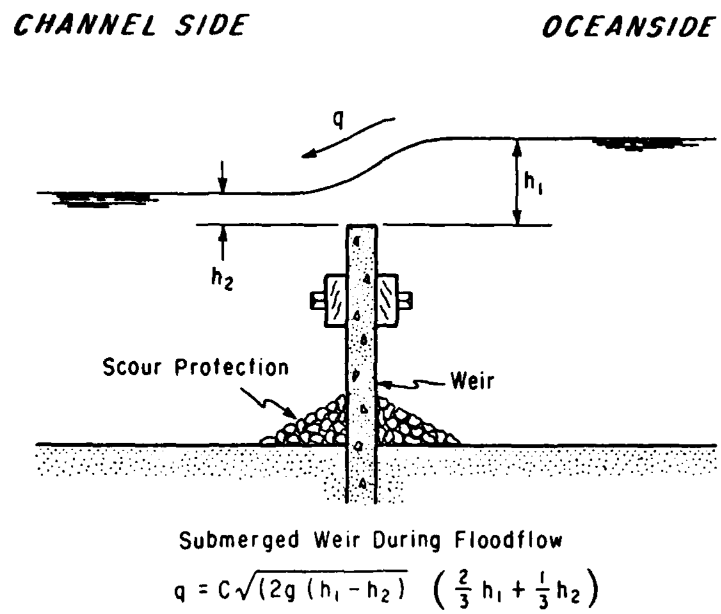

Figure 14. Definition sketch of flow across weir with submerged nappe.

* * * * * * * * * * * * * * * EXAMPLE PROBLEM 1 * * * * * * * * * * * * * * *

GIVEN: The tides on the channel and ocean sides of a weir jetty are as shown in Figure 11.

FIND: Determine the time variation of the discharge per unit width of weir crest if (a) the weir crest is at MTL, and (b) the weir crest is 1.5 feet below MTL. Find the average discharge for each case and the total volume of water per foot of weir length carried across the weir.

SOLUTION: The solution of (a) above (weir crest at MTL) is given in Table 2 and shown graphically in Figure 15. Equation (8) is used to compute the values of q in the table. For example, when $t/T = 0.30$, $h_1 = 0.70$ foot and $h_2 = 0.40$ foot; therefore,

$$q = C\sqrt{2g(h_1 - h_2)}\left(\frac{2}{3}h_1 + \frac{1}{3}h_2\right)$$

$$q = 0.6(4.40)(0.6) = 1.58 \text{ ft}^3/\text{s-ft}$$

The other values of q are similarly computed. Note that for $t/T \geq 0.475$, the water level on the channel side of the weir is above the water level on the oceanside and the direction of flow reverses. The values of h_1 and h_2 are then taken from columns 3 and 2 in Table 2, respectively.

Table 2. Weir discharge calculations for weir crest at MTL.

Time, t/T (fraction of tidal cycle)	h (ocean)[1] (ft)	h (channel)[2] (ft)	q (ft^3/s-ft)
0.25	0	$\leq$ 0	0
0.30	0.70	0.40	1.58
0.35	h_1 1.45	h_2 1.20	3.29
0.40	2.00	1.80	4.16
0.45	2.35	2.30	2.51
0.50	2.50	2.60	-3.90
0.55	h_2 2.45	h_1 2.55	-3.85
0.60	2.10	2.10	0

[1]Obtained from tidal curve for oceanside of weir in Figure 11.

[2]Obtained from tidal curve for channel side of weir in Figure 11.

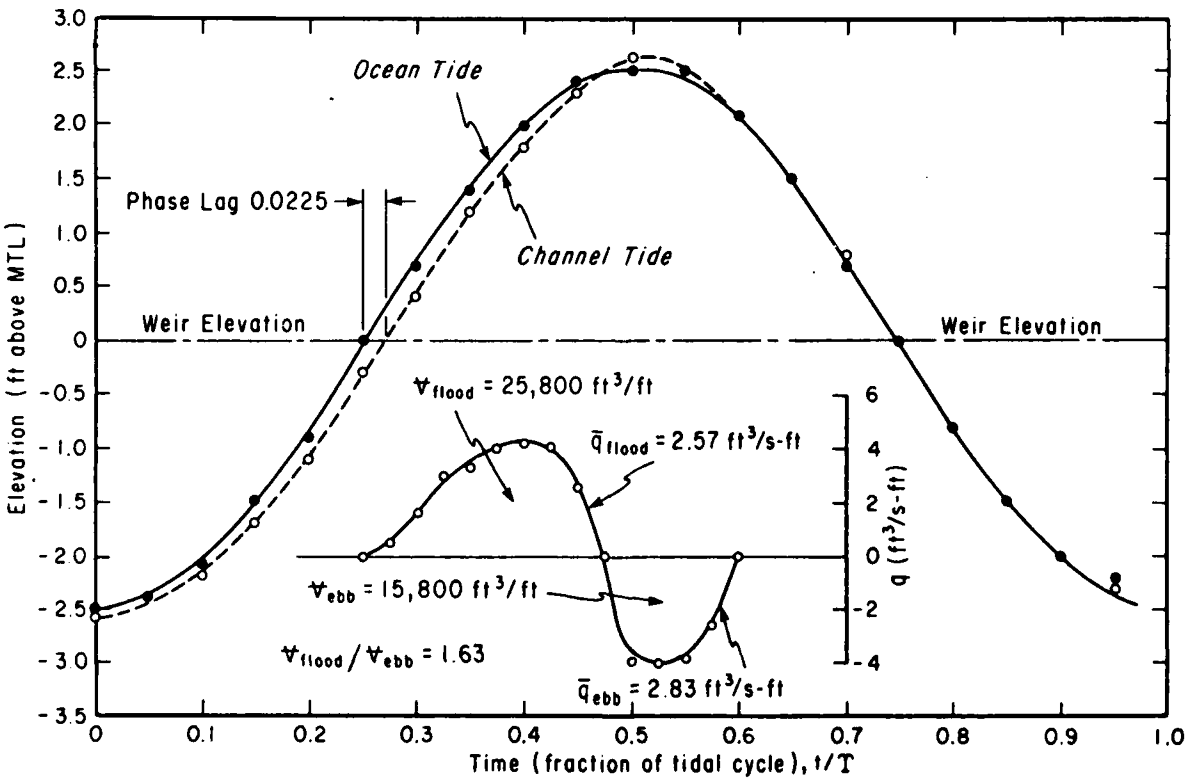

Figure 15. Discharge over weir as a function of time during a tidal cycle (weir 1.5 feet below MTL).

The average floodflow discharge rate is 2.57 cubic feet per second-foot and, from the area under the curve, the flow volume per foot of weir crest during floodtide is 25,800 cubic feet per foot. For ebb flow, the average discharge rate is 2.83 cubic feet per second-foot and the volume is only 15,800 cubic feet per foot. The results for (b) above (weir crest 1.5 feet below MTL) are given in Table 3 and shown graphically in Figure 16. The average discharge rate is 4.50 cubic feet per second-foot during floodtide and 4.52 cubic feet per second-foot during ebbtide. The corresponding volumes per foot of weir length are 65,200 cubic feet per second-foot and 25,200 cubic feet per foot on flood and ebb tides, respectively.

Table 3. Weir discharge calculations for weir crest 1.5 feet below MTL.

Time, t/T (fraction of tidal cycle)	h (ocean)[1] (ft)	h (channel)[2] (ft)	q (ft^3/s-ft)
0.15	0	0	0
0.20	0.70	0.45	1.48
0.25	1.50	1.20	3.59
0.30	h_1 2.20	h_2 1.90	5.54
0.35	2.95	2.70	6.90
0.40	3.50	3.30	7.39
0.45	3.85	3.80	4.12
0.50	4.00	4.10	-6.19
0.55	h_2 3.95	h_1 4.05	-6.11
-0.60	3.60	3.60	0

[1]Obtained from tidal curve for oceanside of weir in Figure 11.

[2]Obtained from tidal curve for channel side of weir in Figure 11.

When the weir-crest elevation is 1.5 feet below MTL there is a larger difference between the inflow and outflow volumes; i.e., 65,200 - 25,200 = 40,000 cubic feet per second-foot as compared with 25,800 - 15,800 = 10,000 cubic feet per second-foot when the weir crest is at MTL.

Changes in the weir elevation, jetty geometry, and inlet hydraulic characteristics will cause changes in the tidal curves on the inlet side of the weir. Since the tidal curves in Figure 11 were obtained in a model test with a weir elevation at MTL, they will not exactly pertain to the condition when the weir is 1.5 feet below MTL. Therefore, the solution to this part of the problem is only an approximation. The problem is determining a priori the tidal curves and phase lag that will be obtained for the jetty geometry and weir elevation of a given inlet. At present, the only way to establish the hydraulic characteristics of an inlet-weir system is to conduct a hydraulic model study. At the same time as the inlet tidal curves are derived from the model, the fraction of the tidal prism entering the inlet across the weir can also be determined.

The volume of water carried over the weir by wave overtopping can be estimated from methods presented in Section 7.22 of the SPM (U.S. Army,

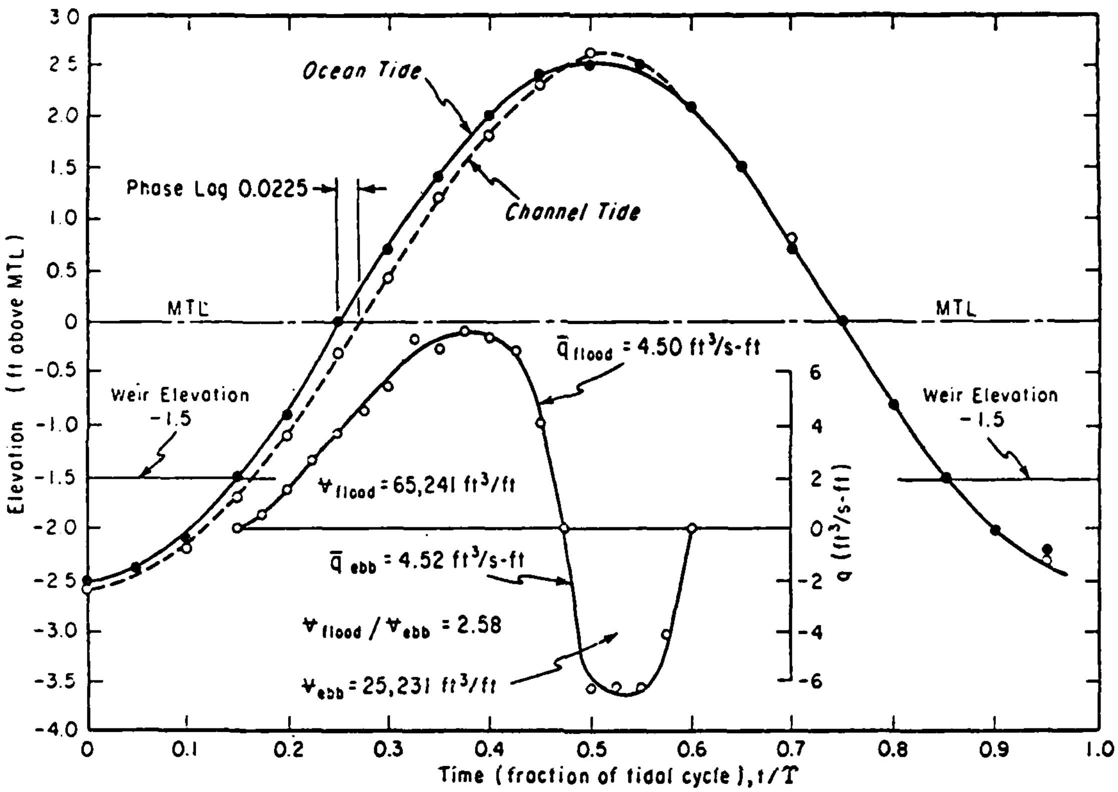

Figure 16. Discharge over weir as a function of time during a tidal cycle (weir 1.5 feet below MTL).

Corps of Engineers, Coastal Engineering Research Center, 1977). This involves determining the time variation of freeboard during a tidal cycle and the variation of water depth along the weir section, and then from these variations computing the overtopping rate. Since the weir section will not normally be perpendicular to the direction of wave approach, the overtopping volume computed should be reduced appropriately. In the absence of specific criteria on which to base an overtopping rate reduction, a reduction factor equal to the square of the cosine of the angle between the weir and incoming wave ray is suggested. Therefore,

$$q' = q \cos^2 \alpha \tag{9}$$

where q' is the reduced overtopping rate, q the overtopping rate if the waves were approaching perpendicular to the weir, and α the angle between the incident wave ray and the axis of the weir.

* * * * * * * * * * * * * * * EXAMPLE PROBLEM 2 * * * * * * * * * * * * * * *

GIVEN: The beach profile along the vertical sheet-pile section of a jetty is as shown in Figure 17. The wave height at the end of the weir is 3.0 feet and the wave period is 7.0 seconds. The tidal curves are as given in Figure 11. The weir crest is at MTL and the waves approach the weir at a 45° angle.

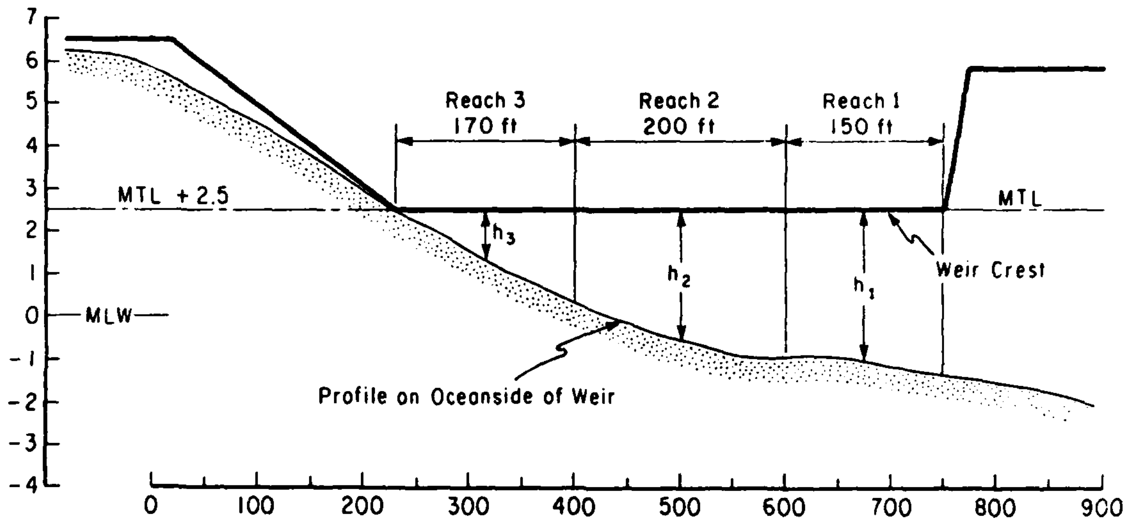

Figure 17. Beach profile adjacent to weir section.

FIND: Determine the wave overtopping rates and the total volume of water carried over the weir by overtopping if wave conditions remain constant over the entire tidal cycle.

SOLUTION: The first step is to subdivide the weir into reaches across which the overtopping rate is assumed constant. For brevity, the example weir is divided into only three reaches. The water depth and wave height at the center of each reach are assumed to determine the overtopping rate in the entire reach. The solution is tabulated for five water depths during a tidal cycle in Table 4. Column 1 gives the water depth at the center of the reach at the indicated time. Column 2 is the length of the reach. At low tide ($t/T = 0$) the shoreline is within reach 2; hence, the length of reach 2 is only 165 feet. Column 3 is the freeboard, the height of the weir at the center of the reach less the water depth. Column 4 is the local wave height. The wave height is depth-limited in most cases and is therefore given approximately by $H_b = 0.78 d_s$ or $H_b = 3.0$ feet, whichever is smaller. Column 5 (obtained from Fig. 7-5 of the SPM) relates breaker height to deepwater wave characteristics assuming a beach slope of 0.07. Columns 6 and 7 are calculated from the tabulated values; column 8 (from Fig. 7-14 of the SPM) was used for the calculations and extrapolated for small values of H_o'/gT^2. Column 9 is the relative freeboard computed from columns 3 and 8. The α and Q_o^* values in columns 10 and 11 are empirical coefficients for use in the SPM overtopping equation (Weggel, 1976) and were obtained from Figure 7-24 of the SPM by interpolating between tabulated points. The values of α and Q_o^* are therefore only approximate. The ranges of d_s/H_o' and H_o'/gT^2 were small, thus α and Q_o^* were assumed constant equal to 0.072 and 0.04, respectively. Column 12 is the overtopping rate per foot of weir crest given by

$$q = \left(g Q_o^* H_o'^3\right)^{1/2} \exp - \left[\frac{0.1085}{\alpha} \log_e \left(\frac{R + h - d_s}{R - h + d_s}\right)\right] \qquad (10)$$

Table 4. Computation of wave overtopping rates for example problem 2.

Reach	(1) d_s (ft)	(2) ℓ (ft)	(3) $h - d_s$	(4) H_b (ft)	(5) H'_o	(6) d_s/H'_o	(7) H'_o/gT^2	(8) R	(9) $h - d_s/R$	(10) α	(11) Q^*_o	(12) q	(13) Q (ft^3/s)
						t/T = 0							
1	1.10	150	2.50	0.86	0.34	3.20	0.0002	1.10	2.27	-----[1]	----	0	0
2	0.60	165	2.50	0.50	0.17	3.50	0.0001	0.65	3.80	-----	----	0	0
3	0	0	----	----	----	----	------	----	----	-----	----	0	0
													0
						t/T = 0.125							
1	2.35	150	1.25	1.83	0.95	2.47	0.0016	1.90	0.66	0.072	0.04	0.974	73.1
2	1.85	200	1.25	1.44	9.69	3.08	0.0019	1.14	1.10	-----	----	0	0
3	0.50	80	1.25	0.39	0.13	3.85	0.0024	0.23	5.43	-----	----	0	0
													73.1
						t/T = 0.25							
1	3.60	150	0	2.81	1.75	2.05	0.0013	3.50	0	0.072	0.04	2.63	197.3
2	3.10	200	0	2.42	1.44	2.15	0.0014	2.88	0	0.072	0.04	1.96	196.0
3	1.25	170	0	0.98	0.44	2.84	0.0018	0.84	0	0.072	0.04	0.33	28.1
													421.4
						t/T = 0.375							
1	4.85	150	−1.25	3.00	1.97	2.46	0.0016	3.84	−0.32	0.072	0.04	8.68[2]	651.0
2	4.35	200	−1.25	3.00	1.97	2.21	0.0014	3.94	−0.32	0.072	0.04	8.45[2]	845.0
3	2.50	250	−1.25	1.95	1.07	2.34	0.0015	2.14	−0.58	0.072	0.04	9.43[2]	1,178.8
													2,674.8
						t/T = 0.50 (high tide)							
1	6.10	150	−2.50	3.00	1.97	3.10	0.0020	3.64	−0.69	0.072	0.04	39.70[2]	------
2	5.60	200	−2.50	3.00	1.97	2.84	0.0018	3.74	−0.67	0.072	0.04	62.00[2]	------
3	3.75	310	−2.50	3.00	1.97	1.90	0.0012	4.14	−0.60	0.072	0.04	25.80[2]	------

[1]No data available.

[2]The values of q are beyond the range of validity of the overtopping equation in the SPM ($h - d_s/R < 0$); they probably overpredict the overtopping rate by a large amount as $h - d_s/R$ becomes much less than 0.

The total overtopping rate for the reach corrected for angle of wave approach is given in column 13. For the example, $Q = q\ell = \ell q \cos^2 \alpha = 0.5 q\ell$ since $\alpha = 45°$ and ℓ = weir-crest length for the reach.

* *

In example problem 2, the overtopping equation (eq. 10) was used for conditions beyond its range of validity since it was used to compute q for cases where $(h - d_s)/R$ is less than zero. The overtopping rates for t/T = 0.375 and t/T = 0.50 are therefore probably too high. At present there is little or no information available to predict overtopping when $(h - d_s)/R$ is less than zero. It seems reasonable to expect overtopping rates to continue to increase for values of $(h - d_s)/R$ slightly less than zero; however, as $(h - d_s)/R$ approaches d_s/R (i.e., as $h \rightarrow 0$), the overtopping rate must approach zero.

The overtopping rate as a function of time through a tidal cycle is shown in Figure 18. A cutoff for the curve was arbitrarily assumed at Q = 600 cubic feet per second. The area under the curve is approximately equal to the volume of water carried over the weir by waves during the tidal cycle. Obviously, if the wave conditions changed during the tidal cycle or if other tidal conditions prevailed, the overtopping volume would be different. The effect of a change in weir length on the total volume of overtopping can also be investigated.

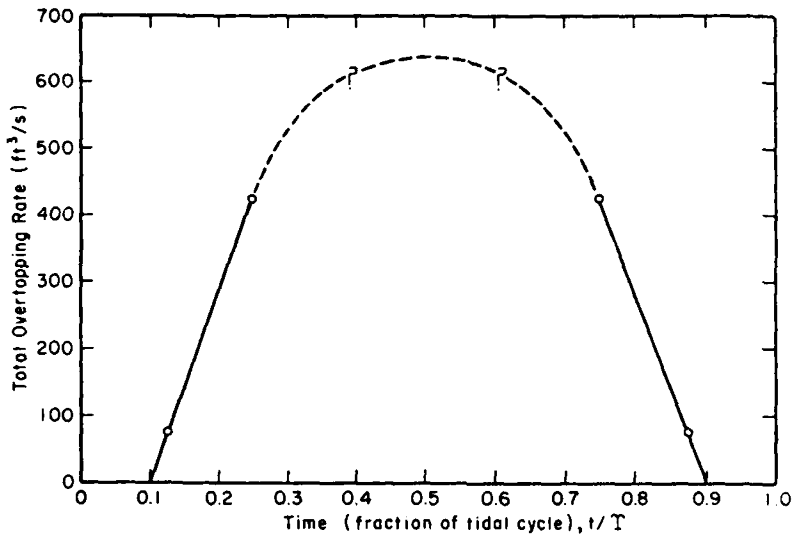

Figure 18. Time variation of overtopping rate.

V. WAVE CONDITIONS IN DEPOSITION BASIN

An important factor in establishing the weir-crest elevation is the level of wave action that can be tolerated in the deposition basin. The allowable level of wave action is dictated by the operating characteristics of the dredge used to perform the bypassing and by the amount of protection required by vessels navigating the channel.

The level of wave action in the deposition basin for a given weir-crest elevation can be estimated from available wave transmission formulas. Assuming no wave energy enters between the jetties and no wave energy passes through the weir section, transmission is by overtopping only; Goda's equation (Goda, Takeda, and Moriya, 1967; Goda, 1969; Seelig, 1976) can then be used, given by

$$\frac{H_t}{H_i} = 0.5 \left\{ 1 - \sin\left[\frac{\pi}{2\alpha} - \left(\frac{h - d_s}{H_i} + \beta\right)\right]\right\} \tag{11}$$

where

H_t = transmitted wave height

H_i = incident wave height

h = height of structure crest above the bottom

d_s = water depth at the structure

α and β are empirical coefficients that depend on the structure's characteristics. Equation (11) is valid for the condition

$$-(\alpha + \beta) \leq \frac{h - d_s}{H_i} \leq (\alpha - \beta)$$

when

$$\frac{(h - d_s)}{H_i} \leq -(\alpha + \beta) \ , \ \frac{H_t}{H_i} = 1.0$$

when

$$\frac{(h - d_s)}{H_i} \geq (\alpha - \beta) \ , \ \frac{H_t}{H_i} = 0$$

For a thin vertical wall, $\alpha = 1.8$ and $\beta = 0.1$, values which apply to a thin sheet-pile weir section. For a vertical-side breakwater with its breadth approximately equal to the water depth, $\alpha = 2.2$ and $\beta = 0.4$. For rubble structures where transmission is by overtopping only, the transmission coefficient, H_t/H_i is (Seelig, 1980)

$$\frac{H_t}{H_i} = \left(0.51 - 0.11 \frac{B}{H}\right) \left(1 - \frac{h - d_s}{R}\right) \tag{12}$$

where B is the crest width of the structure, and R the wave runup height above the stillwater level (SWL) that would occur if the structure crest were above the limit of runup. For a rubble structure, the runup is given by Ahrens and McCartney (1975) as

$$R = \left(\frac{a\xi}{1 + b\xi}\right) H_i \tag{13a}$$

where

ξ = surf parameter given by

$$\xi = \frac{\tan \Theta}{\sqrt{H_i/L_o}} \tag{13b}$$

a, b = empirical coefficients equal to 0.692 and 0.504, respectively, for a structure with two layers oi rubble armor

Θ = angle the seaward face of the weir section makes with a horizontal

L_o = the deepwater wavelength given by $L_o = gT^2/2\pi$ with T the incident wave period and g the acceleration of gravity

When transmission is both through and over the rubble structure, H_t/H_i is given by Seelig (1979) as

$$\frac{H_t}{H_i} = \sqrt{K_o^2 + K_t^2} \tag{14}$$

where K_o is a transmission coefficient for wave energy transmitted by overtopping and K_t a transmission coefficient for wave energy propagated through the structure.

K_o is $(H_t/H_i)^2$ where (H_t/H_i) can be calculated from equation (12). K_t is more difficult to evaluate. Seelig (1979) provides a computer program to calculate the combined transmission coefficient.

Wave heights in the deposition basin vary with the tidal stage as the weir crest submerges and emerges from the water. Maximum wave transmission usually occurs at high tide. Ebb and flood tidal currents flowing across the weir also influence the level of wave action in the deposition area. During flood-flows, the waves are generally lower and longer; during ebb flows, the waves steepen, becoming higher and shorter for the same incident wave conditions.

* * * * * * * * * * * * * * * EXAMPLE PROBLEM 3 * * * * * * * * * * * * * * *

GIVEN: A sinusoidally varying tide with an amplitude of 5.0 feet at a vertical sheet-pile weir. The water depth below MTL at the weir is 7.5 feet. The weir crest is 6 feet above the bottom (1.5 feet below MTL). The wave height and period are H_b = 6.0 feet and T = 8.0 seconds.

FIND: The wave height variation in the deposition basin over a tidal cycle assuming waves approach the weir perpendicularly.

SOLUTION: The time history of water level at the weir is shown in Figure 19. For an impermeable sheet-pile weir, equation (11) with $\alpha = 1.8$ and $\beta = 0.1$ can be used. The transmission coefficient is given by

$$\frac{H_t}{H_i} = 0.5 \left\{ 1 - \sin \left[\frac{\pi}{2\alpha} \left(\frac{h - d_s}{H_i} + \beta \right) \right] \right\}$$

$$\frac{H_t}{H_i} = 0.5 \left\{ 1 - \sin \left[\frac{\pi}{3.6} \left(\frac{6.0 - d_s}{H_i} + 0.1 \right) \right] \right\}$$

since h = 6.0 feet and $2\alpha = 3.6$. The solution is given in Table 5 and is presented graphically in Figure 20. Maximum wave transmission occurs at high tide (d_s = 10.0 feet) with H_t = 4.42 feet. Table 5 is calculated by first determining the incident wave height. Since the water depth at the weir is only 5 feet, the 6-foot-high incident wave will break seaward of the weir. The maximum wave height that can occur at the weir is given approximately by the condition that $H_i \leq 0.78\ d_b$. Therefore, H_i = 0.78 (5.0) = 3.90 feet. Substituting into equation (11)

$$\frac{H_t}{H_i} = 0.5 \left\{ 1 - \sin \left[\frac{\pi}{3.6} \left(\frac{6.0 - 5.0}{3.90} + 0.1 \right) \right] \right\}$$

$$\frac{H_t}{H_i} = 0.5\ [1 - \sin(0.3110)] = 0.347$$

Thus, $H_t = 0.347\ H_i = 0.347(3.90) = 1.35$ feet. Note that equation (11) is valid even though the water level is 1 foot below the weir crest. The equation is equally valid for conditions when d_s is greater than h. (See Seelig, 1976.)

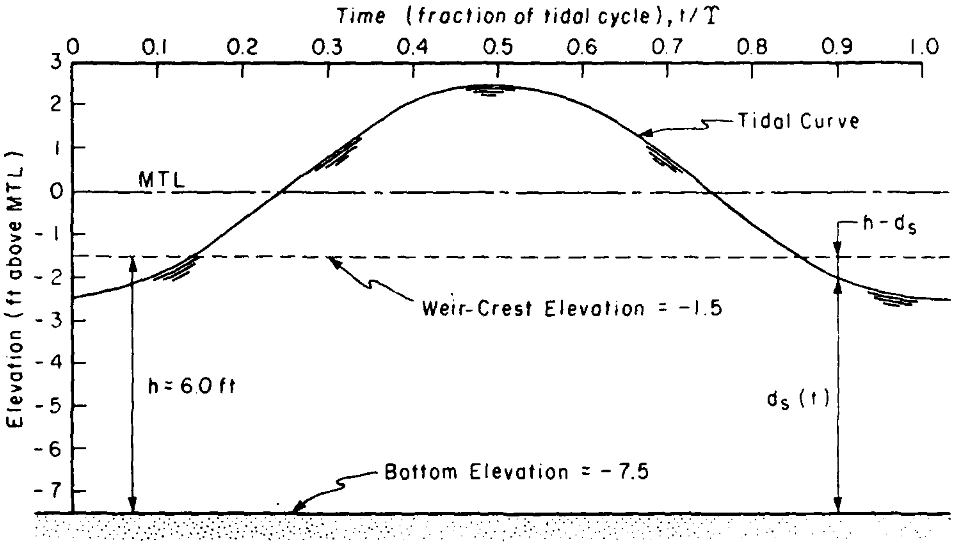

Figure 19. Water level at weir as a function of time.

Table 5. Computation of wave transmission by overtopping for example problem 3.

Row	Time (fraction of tidal period)	Depth (ft)	H_i (ft)	H_t/H_i	H_t (ft)
1	0	5.00	3.90[1]	0.347	1.35
2	0.05	5.10	3.98[1]	0.360	1.43
3	0.10	5.50	4.26[1]	0.406	1.73
4	0.15	6.00	4.68[1]	0.456	2.14
5	0.20	6.70	5.23[1]	0.514	2.69
6	0.25	7.50	5.85[1]	0.568	3.32
7	0.30	8.20	6.00	0.615	3.69
8	0.35	8.95	6.00	0.667	4.00
9	0.40	9.50	6.00	0.704	4.23
10	0.45	9.85	6.00	0.727	4.36
11	0.50	10.00	6.00	0.737	4.42
12[2]	0.55	9.85	6.00	0.727	4.36

[1] Waves are depth-limited and break seaward of weir; incident wave heights are given approximately by $H_i = 0.78\ d$.

[2] Because of tidal curve symmetry, solution is symmetric about t = 0.50.

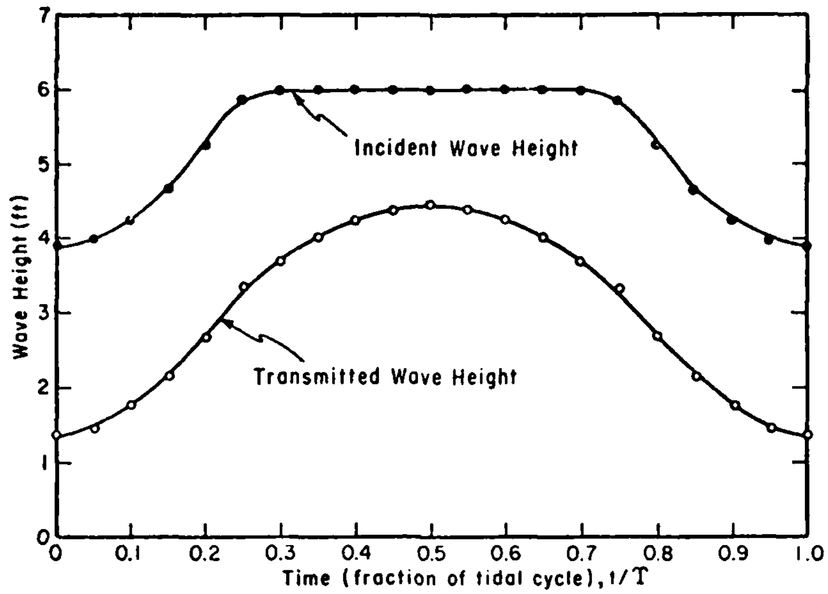

Figure 20. Time variation of incident and transmitted wave heights.

* * * * * * * * * * * * * * * EXAMPLE PROBLEM 4 * * * * * * * * * * * * * * *

GIVEN: The same conditions given for the preceding example; however, a rubble weir is used with a crest width of 15 feet and a side slope of 1 on 2.

FIND: The wave height variation in the deposition basin over a tidal cycle assuming the waves approach perpendicular to the weir.

SOLUTION: For a rubble weir, equations (12) and (13) can be used with $a = 0.692$ and $b = 0.504$. The transmission coefficient is given by

$$\frac{H_t}{H_i} = \left(0.51 - 0.11 \frac{B}{h}\right)\left(1 - \frac{h - d_s}{R}\right)$$

with R given by equation (13a)

$$R = \left(\frac{a\xi}{1 + b\xi}\right) H_i$$

with (eq. 13b)

$$\xi = \frac{\tan \Theta}{\sqrt{H_i / L_o}}$$

Results of the calculations are given in Table 6; the solution is presented graphically in Figure 21. Maximum wave transmission occurs at high tide with $H_t = 2.55$ feet. Note that transmitted wave heights for the rubble weir are significantly lower than transmitted wave heights for the sheet-pile weir of the preceding example.

Table 6. Computation of wave transmission by overtopping for example problem 4.

Row	Time (fraction of tidal period)	Depth (ft)	H_i (ft)	ξ	R (ft)	H_t/H_i	H_t (ft)
1	0	5.00	3.90[1]	4.58	3.74	0.096	0.37
2	0.05	5.10	3.98[1]	4.54	3.80	0.107	0.43
3	0.10	5.50	4.26[1]	4.39	4.02	0.146	0.62
4	0.15	6.00	4.68[1]	4.28	4.36	0.190	0.89
5	0.20	6.70	5.23[1]	3.96	4.78	0.241	1.26
6	0.25	7.50	5.85[1]	3.74	5.24	0.290	1.70
7	0.30	8.20	6.00	3.70	5.36	0.328	1.97
8	0.35	8.95	6.00	3.70	5.36	0.368	2.21
9	0.40	9.50	6.00	3.70	5.36	0.398	2.39
10	0.45	9.85	6.00	3.70	5.36	0.417	2.50
11	0.50	10.00	6.00	3.70	5.36	0.425	2.55
12[2]	0.55	9.85	6.00	3.70	5.36	0.417	2.50

[1]Waves are depth-limited and break seaward of weir; incident wave heights are given approximately by $H = 0.78\ d$.

[2]Because of tidal curve symmetry, solution is symmetric about $t = 0.50$.

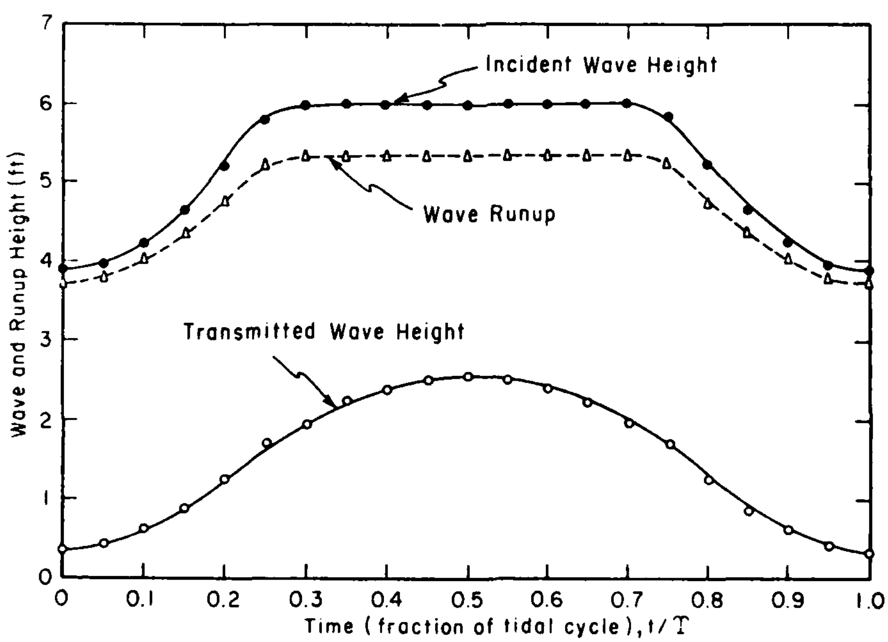

Figure 21. Time variation of incident and transmitted wave heights.

* *

VI. EFFECT OF PROJECT ON SEDIMENT BUDGET

1. Preproject Sediment Budget.

To develop an operating plan and establish bypassing needs for a proposed weir jetty project, a thorough understanding of inlet processes at the project site is necessary, particularly the prevailing sand transport conditions. A preproject sediment budget for the inlet should be constructed from all available data, including (a) aerial photography; (b) beach profiles updrift and downdrift of the inlet; (c) beach nourishment records; (d) inlet dredging records; (e) hydrographic surveys of the inlet, shoals, and ocean bar; (f) results of dye and drifter studies; (g) wave data (to determine longshore sand transport rates); and (h) transport rates measured at nearby locations. Wave data which include direction may be available from hindcasts, Summaries of Synoptic Meteorological Observations (SSMO); or visual observations. Several alternative postproject sediment budgets may subsequently be evaluated to establish an optimum operating procedure for the system.

The development of a sediment budget is best illustrated by an example. Jarrett (1976) constructed a sediment budget for Bogue Banks, Shackleford Banks, and Beaufort Inlet, an area just west of Cape Lookout in Brunswick County, North Carolina. Data available and developed for his beach study were surveys of the inlet dating back to 1939, beach recession rates, beach nourishment records, inlet dredging records, hydrographic surveys of inlet shoals, SSMO wave data, refraction analyses, and information on local sea level rise. A conceptualized layout of the area is shown in Figure 22. Jarrett developed information on sand volume changes occurring in the three elements of the sediment budget (Bogue Banks, Shackleford Banks, and Beaufort Inlet). The conservation of sand for each element was expressed as

$$Q_{in} - Q_{out} = \frac{\Delta V\!\!\!\!-}{\Delta t} \qquad (15)$$

where

Q_{in} = volumetric sediment inflow rate

Q_{out} = volumetric sediment outflow rate

$\Delta V\!\!\!\!-$ = change of sediment volume contained in the element

Δt = time interval over which the change took place

The inflow of sediment to each element, $Q_{in}\,\Delta t$, less the sediment carried out of the element, $Q_{out}\,\Delta t$, must balance the change in sediment volume contained within the element $\Delta V\!\!\!\!-$. The sources of sediment gain and loss must be identified. For Bogue Banks (28,000 feet long), measured beach profiles shifted landward at an average rate of 4.2 feet per year between 1936 and 1974, the beginning and ending times of the example sediment budget. Data on beach changes were obtained from surveys and analyses of aerial photos. The 4.2-foot average beach recession rate converts to a change in volume equal to -4.2 feet per year × 28,000 feet × 1.3 cubic yards per square foot = -153,000 cubic yards per year. The factor 1.3 cubic yards per square foot (specific for this

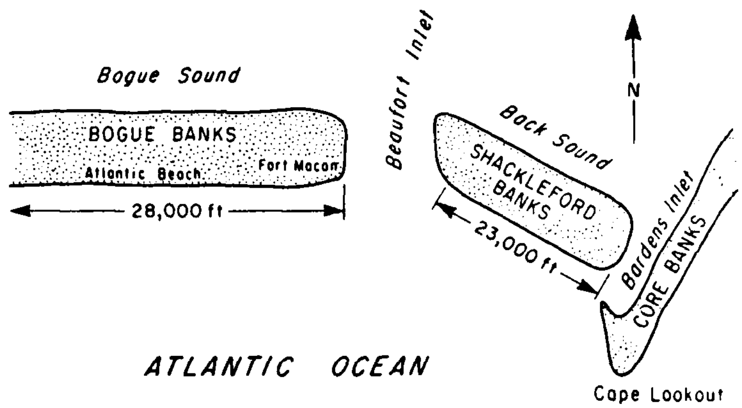

Figure 22. Conceptual layout of Bogue Banks, Beaufort Inlet, and Shackleford Banks, North Carolina.

site) indicates that a volume change of about 35 cubic feet of sand on the profile is needed to cause a 1-square-foot change in beach area (1-foot recession along 1 foot of beach). This value will vary from site to site but can be obtained from the average depth of closure of beach profiles taken at various times at a given site. The shoreline recession on Shackleford Banks was -8.2 feet per year, corresponding to a volume loss of -245,000 cubic yards per year along the 23,000 feet of shoreline.

Changes in sand volume on the ocean bar between 1936 and 1974 determined from hydrographic surveys amounted to a total sand loss of 11,750,000 cubic yards or an annual rate of -309,000 cubic yards per year. Records indicate that hopper dredging removed 23,920,000 cubic yards of sand from the inlet between 1936 and 1974, giving an annual loss rate of -629,000 cubic yards per year. From hydrographic surveys, accumulations in the bay area behind the inlet amounted to 58,000 cubic yards per year in Back Sound and 134,000 cubic yards per year in Bogue Sound for a combined rate of -192,000 cubic yards per year.

Two beach nourishment projects were completed on Bogue Banks between 1936 and 1974; 92,800 cubic yards was placed in 1965 and 105,000 cubic yards in 1969. If the nourishment had been placed at a uniform rate over the 1936-74 period it would have averaged 5,000 cubic yards per year. Sand losses offshore are assumed to result from sea level rise and can be estimated using Bruun's (1962) method (see also Weggel, 1979). For Bogue Banks, 32,000 cubic yards per year is lost offshore; Shackleford Banks loses 33,000 cubic yards per year.

The amount of sand in transport along the beaches driven by wave-induced longshore currents was estimated from SSMO wave data (ship observations) brought from deep water to shore using a refraction analysis. The coefficient of proportionality relating longshore sand transport, Q_ℓ, with longshore wave energy flux factor, $P_{\ell s}$, can be taken as one of the three unknowns in the system of three simultaneous equations that result from the sediment budget. All values of longshore transport can be expressed as a fraction of the eastward longshore transport at the east end of Bogue Banks. A schematic of the inflow and outflow of sediments from the three elements of the sediment budget is presented in Figure 23. The three unknowns defined in the figure are: Q_E, the eastward longshore sand transport rate at the east end of Bogue

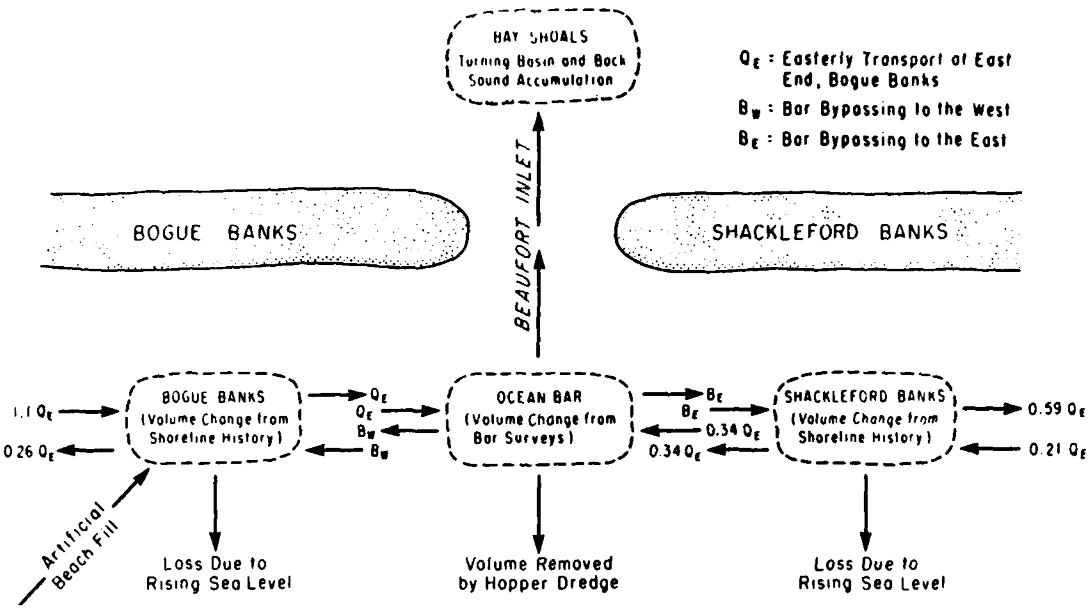

Figure 23. Schematized sediment budget analysis for Beaufort Inlet, North Carolina (from Jarrett, 1976).

Banks (Note that all longshore sand transport rates are expressed as a constant times this transport rate; hence, the eastward longshore transport rate at the west end of Bogue Banks is 1.1 Q_E while the westward rate is 0.26 Q_E, etc.); B_W, the amount of sand bypassed to the west from the ocean bar; and B_E, the amount of sand bypassed to the east from the ocean bar. The following three equations, one for each element, can be written expressing conservation of sand in the system (refer to Fig. 23)

$$1.1\ Q_E - Q_E - 0.26\ Q_E + B_W + 5 - 32 = -153 \text{ (Bogue Banks)}$$

$$0.21\ Q_E + B_E - 0.34\ Q_E - 0.59\ Q_E - 33 = -245 \text{ (Shackleford Banks)}$$

$$Q_E + 0.34\ Q_E - B_W - B_E - 192 - 629 = -309 \text{ (Beaufort Inlet)}$$

where the quantities in the equations are in thousands of cubic yards per year. Solving these simultaneous equations, the three unknowns are (in cubic yards per year)

$$Q_E = 378{,}000$$

$$B_W = -66{,}000$$

$$B_E = 60{,}000$$

The minus sign for B_W indicates that transport is in the opposite direction from that assumed in Figure 23.

The resulting sediment budget is the long-term average disposition of sediments in the region. Sediment disposition in any given year may differ from the results of the preceding analysis because of variations in wave conditions, the occurrence of unusual storms, etc. Consequently, sediment budgets for other time intervals should be constructed, data permitting. Details of the variation in the sediment budget for Beaufort Inlet are provided in Jarrett (1976).

2. Postproject Sediment Budget.

After construction of a sand-bypassing system, the amount of sand transferred from the updrift to downdrift beach can be controlled by the amount of dredging performed; consequently, the amount of bypassing is no longer an unknown in the sediment budget but depends on the operation plan for the project. Likewise, the amount of sand entering and leaving the system at the extreme ends of the adjacent beaches can, as a first approximation, be assumed to be unaffected by inlet modifications. The Beaufort Inlet example is used to illustrate four project operation strategies (see Fig. 24). It is assumed that the net amount of sand accumulating in the inlet is zero and that bypassing is from west to east (i.e., from Bogue Banks to Shackleford Banks). The optimum system is one that will keep sediment from the navigation channel, minimize any adverse effects of the navigation structures on adjacent beaches (to preclude updrift or downdrift beach erosion), and minimize the amount of dredging required to bypass or backpass (pump sand from the inlet to updrift beaches) sand.

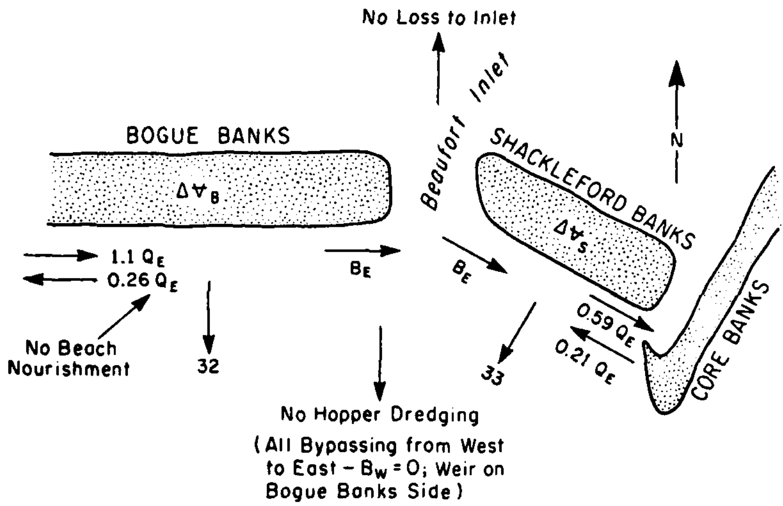

Figure 24. Typical sediment budget conditions after weir-jetty construction.

a. Strategy 1. A first bypassing strategy at Beaufort Inlet might be to limit updrift beach losses to those that result from sea level rise (i.e., losses that occur naturally along the beach away from the influence of the inlet) which specifies that the amount of sand lost from Bogue Banks is 32,000 cubic yards per year. The volume change occurring on Shackleford Banks, ΔV_s, and the amount of sand to be bypassed, B_E, become the unknowns in the resulting system of two simultaneous equations (see Fig. 25). The equation for Bogue Banks is

$$415.8 \text{ (gain from west)} - 98.3 \text{ (loss to west)} - 32 \text{ (lost offshore)} - B_E = -32 \text{ (net volume lost)}$$

and for Shackleford Banks is

$$B_E - 33 \text{ (lost offshore)} - 223.0 \text{ (lost to east)} + 79.4 \text{ (gain to east)} = \Delta V_s$$

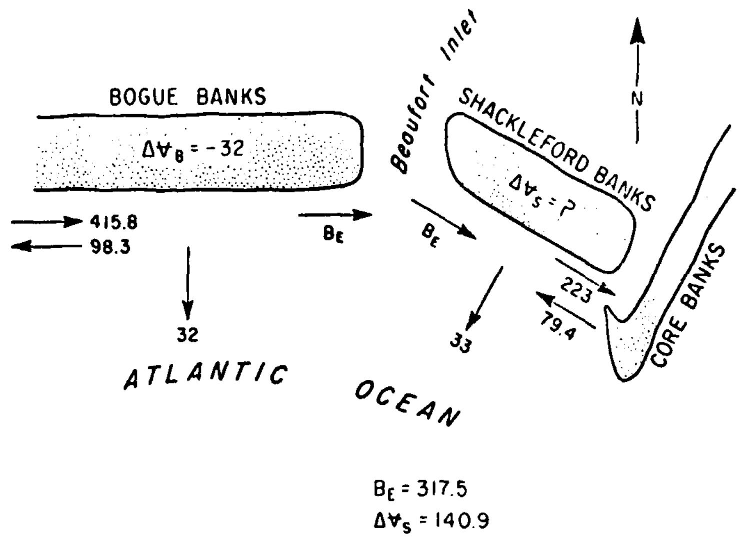

Figure 25. Weir-jetty construction sediment budget for strategy 1.

The values of the unknowns are

$$B_E = 317.5 \quad \text{and} \quad \Delta V_S = +140.9$$

Consequently, if this strategy is adopted, Shackleford Banks will accumulate sand at the rate of 140,900 cubic yards per year, corresponding to a shoreline advance of 4.7 feet per year if the sand were uniformly distributed along the beach by wave action. The amount of sand to be bypassed is 317,500 cubic yards per year, a relatively large volume. The volume of sand that must be bypassed under strategy 1 may exceed the ability of the waves to redistribute it along the downdrift beach, which could cause problems in identifying needed disposal areas.

b. Strategy 2. A second strategy (Fig. 26) might be to limit downdrift beach losses to those resulting from sea level rise so that ΔV_S is fixed at -33,000 cubic yards per year. The Bogue Banks equation is then

$$415.8 \text{ (gain from west)} - 98.3 \text{ (loss to east)} - 32 \text{ (lost offshore)}$$
$$- B_E = \Delta V_B$$

and the Shackleford Banks equation becomes

$$B_E - 223.0 \text{ (loss to east)} + 79.4 \text{ (gain from east)}$$
$$- 33 \text{ (lost offshore)} = -33 \text{ (net volume lost)}$$

with the solution

$$B_E = 143.6 \quad \text{and} \quad \Delta V_B = +141.9$$

The amount of sand to be bypassed is small but the updrift beach adjacent to the weir will accumulate 141,900 cubic yards of sand per year and cause a seaward movement of the beach at a minimum rate of 3.9 feet per year if the sand were spread uniformly along Bogue Banks. Localized areas of accretion could advance faster. It is doubtful whether a weir jetty can operate according to

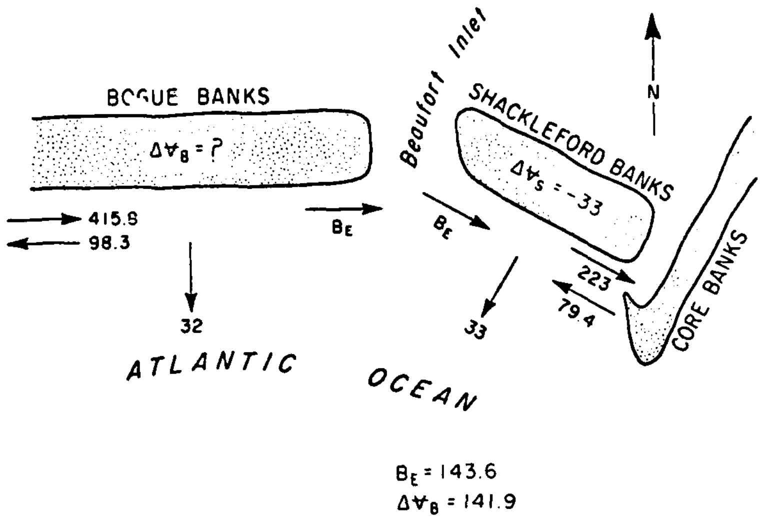

Figure 26. Weir-jetty construction sediment budget for strategy 2.

the assumptions made under this strategy since the weir will stabilize the updrift beach and any excess sand will be transported over the weir into the deposition basin where it will eventually require dredging.

c. Strategy 3. A third strategy (Fig. 27) might distribute losses or gains equally over both updrift and downdrift beaches, i.e., $\Delta V_S = \Delta V_B$. (Another possible alternative is to equalize beach recession or accretion rates.) The Bogue Banks equation is then

$$415.8 \text{ (gain from west)} - 98.3 \text{ (loss to west)} - 32 \text{ (lost offshore)} - B_E = \Delta V_B$$

and the Shackleford Banks equation is

$$B_E - 223.0 \text{ (loss to east)} + 79.4 \text{ (gain from east)} - 33 \text{ (lost offshore)} = \Delta V_S = \Delta V_B$$

and the solution is

$$B_E = 231.0 \quad \text{and} \quad \Delta V_S = \Delta V_B = 54.5$$

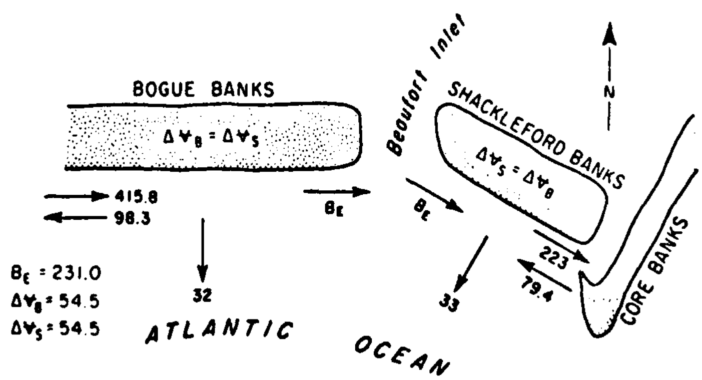

Figure 27. Weir-jetty construction sediment budget for strategy 3.

Both the updrift and downdrift beaches gain sand so that the updrift beach (Bogue Banks) will advance at a rate of 1.5 feet per year and the downdrift beach (Shackleford Banks) will advance at 1.8 feet per year. The problem is the same with this strategy as with the preceding one--the beach updrift of the weir is stabilized by construction of the weir and any extra sand is carried into the deposition basin. It will not be possible to limit accumulation in the deposition basin to 231,000 cubic yards per year.

d. Strategy 4. A fourth strategy (probably the most realistic one for operating a weir-jetty system) is to keep losses from the updrift beach at zero to stabilize the updrift beach and generally ensure quasi-steady state operation of the weir (see Fig. 28). A disadvantage of this strategy is that it might require a greater volume of sand to be transferred than other strategies. Under strategy 4 the volume change on Bogue Banks, $\Delta\forall_B$, is taken as zero and the Bogue Banks equation becomes

$$415.8 \text{ (gain from west)} - 98.3 \text{ (loss to west)} - 32 \text{ (lost offshore} - B_E = 0$$

and the Shackleford Banks equation is

$$B_E - 223.0 \text{ (loss to east)} + 79.4 \text{ (gain from east)} - 33 \text{ (lost offshore)} = \Delta\forall_S$$

Solution of the two equations gives

$$B_E = 285.5 \quad \text{and} \quad \Delta\forall_S = 108.9$$

The rate of accumulation on the downdrift shoreline (Shackleford Banks) is thus 108,900 cubic yards per year and the shoreline moves seaward at 3.6 feet per year if the accumulation is uniformly distributed along the shoreline. The amount of sand to be bypassed is 285,500 cubic yards per year, the volume that would probably accumulate in the deposition basin if the updrift beach is stabilized by construction of a weir jetty with a sandtight landward section and a weir section positioned to establish a dynamically stable beach planform.

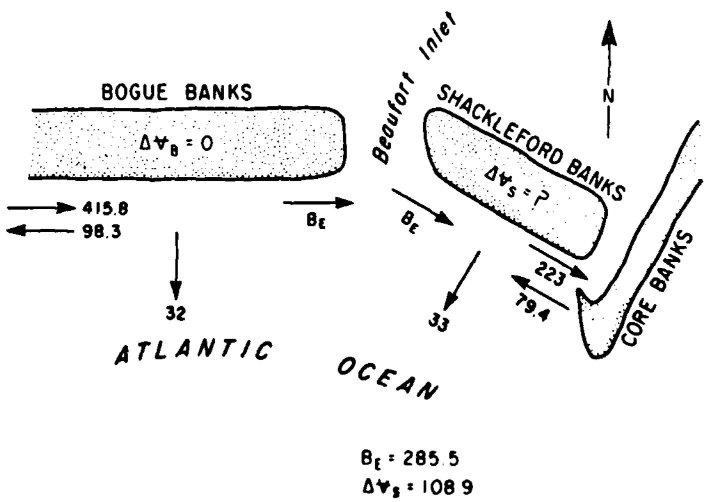

Figure 28. Weir-jetty construction sediment budget for strategy 4.

The wave climate and consequently the longshore transport rates at a site can vary significantly from year to year. If data are available for other time periods that represent extremes in transport climate, additional sediment budgets should be constructed to provide further insight into the range of conditions under which a weir-jetty system is expected to operate.

The Beaufort Inlet example may be atypical since it describes an area which has a net increase in sand accumulation. For example, the problem is one of distributing excess sand in the overall system between updrift and downdrift beaches. In other cases, allocating a sand deficit between updrift and downdrift beaches at a reasonably stable condition appears to be necessary for the successful performance of a weir jetty. Seasonal fluctuations in transport conditions at a weir are discussed in Section VII which deals with predicting the storage requirements of weir-jetty deposition basins.

VII. UPDRIFT BEACH AND DEPOSITION BASIN STORAGE ANALYSIS

In designing the updrift beach, an assessment must be made of the expected equilibrium shoreline under various directions of wave attack, particularly if seasonal reversals in transport are common at the site. At sites where reversals are common, an optimum weir-jetty system will store sand in the updrift beach to be transported by waves back up the beach, thus precluding erosion of the updrift beach and the need for backpassing. This is considered "active" storage (see Fig. 29). Sand will also be stored adjacent to the updrift jetty where it is sheltered by the jetty and cannot be removed by normal wave action. This sand, considered "dead" storage (Fig. 29), accumulates after construction of the jetty and once deposited tends to remain in place. The amount of active storage needed in the updrift beach depends on the magnitude and frequency of reversals in transport. If reversals are large in magnitude and occur seasonally with long periods of transport in one direction followed by long periods of transport in the other direction, the amount of active storage required may be large. On the other hand, if reversals are frequent with short periods of time between them, the amount of active storage required will be small.

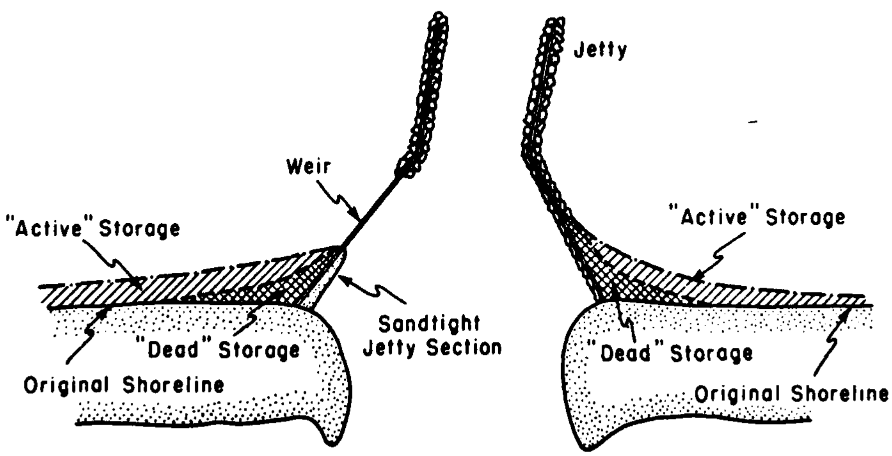

Figure 29. Sand storage on updrift and downdrift beaches near a weir jetty.

The amount of sand that needs to be stored in the updrift fillet can be estimated from a mass curve constructed from the time history of longshore transport at the site. Such a time series can be constructed from wave data obtained by LEO observations, SSMO, or wave hindcast data. A representation of the time series is shown in Figure 30. The upper curve in the figure represents the time history of the longshore transport rate; the lower mass curve is the integral under the longshore transport rate curve and represents the cumulative amount of sand passing the observation point from the time observations began. The line superimposed on the mass curve is a best-fit straight line; its slope represents the net longshore transport rate at the site. The deviation of the mass curve from the straight line corresponds to the amount of sand needed in active storage to nourish updrift beaches during reversals in transport. Figure 31 represents a similar curve for a site where reversals are more frequent in contrast with a site where reversals are seasonal. When reversals are short term and frequent, the amount of active storage required in the updrift beach is generally less and the updrift beach will be less variable in planform. There is less updrift shoreline retreat during periods of transport reversal since the duration of reversals is shorter.

The minimum amount of sand to be transferred and the capacity of the deposition basin can also be established from the mass curve. If bypassing is performed biannually, the ordinate of the straight line on the mass curve (net or average transport) at t = 2 years will give the minimum amount of deposition basin storage required. It also represents the minimum amount of sand to be bypassed after a 2-year period. In addition, the mass curve provides information on scheduling bypassing operations. If bypassing to the downdrift beach is performed when the updrift beach is emptying (the trend of the slope of the mass curve is negative), transport will be in the updrift direction and sand placed on the downdrift beach will move toward the inlet. Bypassing should be scheduled for those seasons when the trend of the mass curve slope is positive to ensure that bypassed sand moves downcoast away from the inlet.

A major problem in constructing the required mass curve is the availability of sufficient, reliable wave data to develop the time series of longshore transport rates. Because the wave climate at a site may vary from year to year, 1 or 2 years of wave records may not be enough to adequately define the magnitude and duration of reversals. A minimum of 3 years of wave data should be used and even then, conditions in any 1 year might differ appreciably from conditions during the period of record. The designer should investigate conditions that deviate from measured records to determine project performance under extreme conditions. Questions such as, "How will the project perform if the net longshore transport rate has been underestimated or overestimated, or if the project experiences an extreme storm?" should be asked and the consequences evaluated.

VIII. WEIR SECTION LENGTH

In general, weir section length should be established to extend seaward beyond the normal breaker line. Most of the sand transported over the weir moves across in a relatively narrow region close to where the weir, beach, and waterline intersect. Preliminary results from laboratory tests indicate that sand transported over the weir in this region moves as bedload. The amount of sand transported varies with wave conditions and tidal stage. There is also

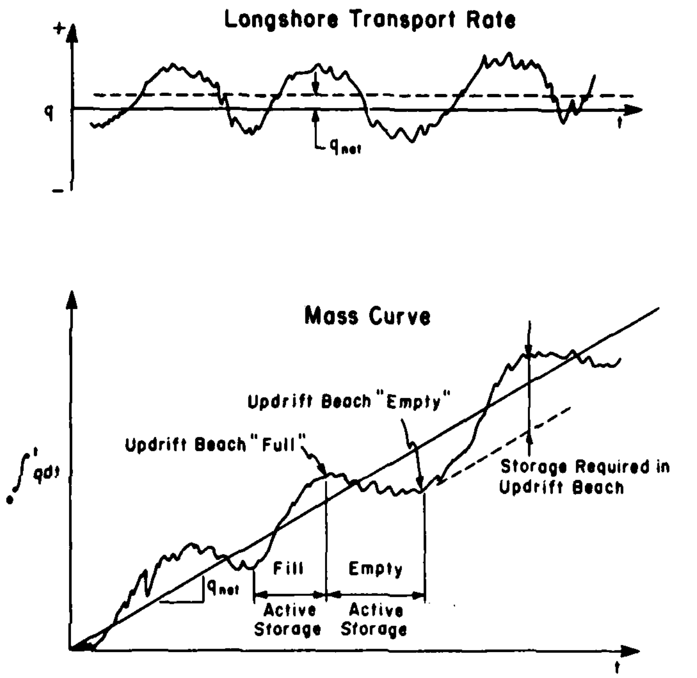

Figure 30. Time history of longshore transport rate and mass curve.

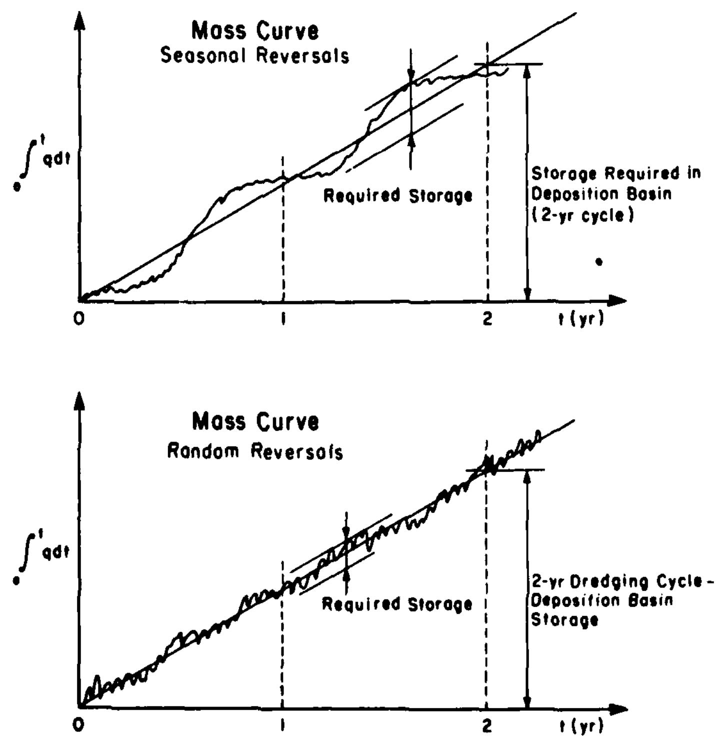

Figure 31. Longshore transport mass curves for a site where reversals are seasonal and a site where reversals are random.

some evidence that farther from shore in deeper water near the breaker region, sand moves across the weir in suspension. In the laboratory the amount in suspension appears relatively small in comparison with the amount carried over as bedload near shore. At Perdido Pass, Mississippi, in the Gulf of Mexico, an offshore bar that parallels the updrift beach, and which appears to be continuous across the weir to the channel side of the jetty, suggests that sufficient transport occurs offshore as suspended load to maintain the bar across the weir. Where offshore bars occur, the weir should probably extend seaward beyond the bars. The length of the weir in this case can be determined from normal beach profiles that exist updrift of the proposed jetty away from the influence of the inlet. Seasonal changes in the profiles should be taken into consideration. Figure 32 shows how weir length can be established from beach profiles exhibiting prominent offshore bars. Where offshore bars are not prominent, the seaward end of the weir section should be seaward of the normal breaker location. The profile used to establish breaker location should be the profile expected after construction of the project, not the profile existing at the unimproved inlet. An estimate of the postconstruction profile can be obtained by examining existing profiles updrift of the proposed jetties, far enough away from the inlet to avoid being influenced by tidal currents and localized inlet wave refraction. Based on current knowledge, there are no definitive guidelines for selecting the wave height to determine the breaker depth; however, a first estimate can be obtained by using the average annual significant breaker height at the site. (Values for the average annual breaker height at various U.S. coastal locations are given in Ch. 4 of the SPM.) Obviously, tidal stage also influences breaker location with respect to the weir. Waves of a given height will break farther seaward from a fixed point on the shore at low tide than at high tide. To ensure that the average annual significant breaker height occurs landward of the seaward end of the weir section, the water level used for the analysis should be MLW. At low tide, waves larger than the average annual significant breaker height break seaward off the end of the weir section; however, at water levels above MLW these larger waves may break adjacent to the weir section. Also, during storms when larger waves occur, storm surge often raises the water level, allowing larger waves to transport suspended sand across the weir.

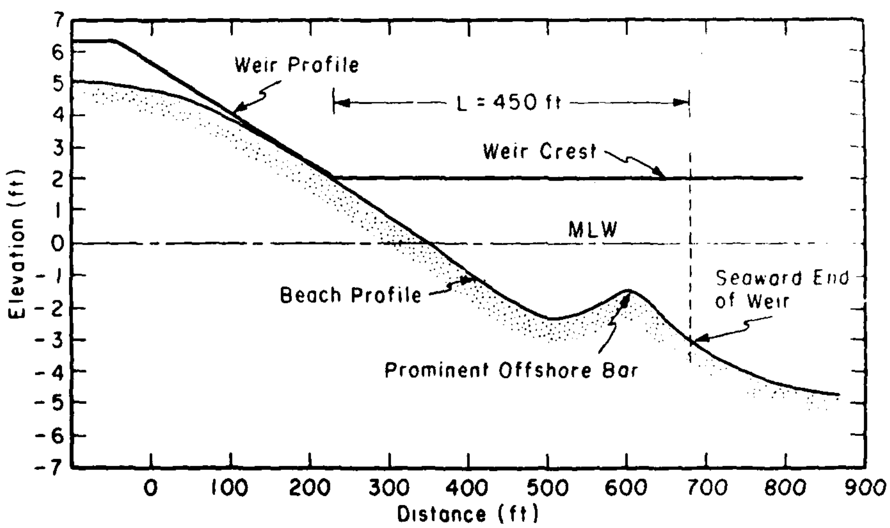

Figure 32. Weir length in the presence of a well-defined offshore bar.

* * * * * * * * * * * * * * * EXAMPLE PROBLEM 5 * * * * * * * * * * * * * * *

GIVEN: The beach profile updrift of an inlet where a weir jetty is proposed is shown in Figure 33. The weir elevation is at MTL. The normal tidal range is 4.0 feet and the average annual significant breaker height is 3.0 feet.

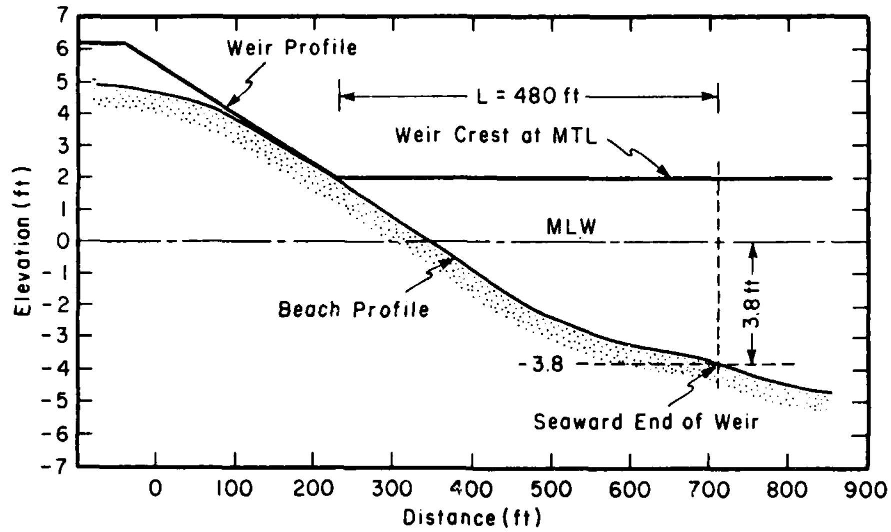

Figure 33. Weir length as established by average annual breaker height (final beach profile after weir construction is assumed to be similar to preconstruction profile).

FIND: Estimate the length of the weir section and the height of the breaker at the end of the weir section at mean high water (MHW).

SOLUTION: The water depth in which a 3.0-foot-high wave will break is given approximately by the solitary wave theory expression

$$\frac{H_b}{d_b} = 0.78 \qquad (16)$$

(The influence of local beach slope on the ratio H_b/d_b can be considered by using the design curves in Ch. 7 of the SPM; however, eq. 16 is accurate enough for determining weir length.) The breaking depth is

$$d_b = \frac{H_b}{0.78} = 1.28(3.0) = 3.8 \text{ feet}$$

The point where the beach profile is 3.8 feet below MLW locates the approximate seaward end of the weir section. The length of the weir is therefore 480 feet. When the water level is at MHW, the depth at the seaward end of the weir is 3.8 + 4.0 = 7.8 feet. The breaker height in 7.8 feet of water (from eq. 16) is

$$H_b = 0.78 d_b = 0.78(7.8) = 6.1 \text{ feet}$$

* *

There has been some concern about possible "sanding-in" of short weir sections. Sanding-in was believed to be possible during severe storms when large amounts of sand might be transported along the updrift beach to the weir (but not carried over the weir) and cause rapid deposition adjacent to the updrift side of the weir. The landlocked weir would then cease to pass sand, requiring excavation of the accumulated sand before the weir could again function. Because of this concern, existing weir sections have been extended to more than 1,000 feet long. However, experience with existing weir jetties has indicated that sanding-in may not be a significant problem; no existing weirs have sanded-in. In fact, the shoreline of the updrift beach has not moved seaward much beyond the landward end of the weir section although the profile adjacent to the weir may have flattened. Since most of the transport appears to be over the landwardmost end of the weir, lengths in excess of 500 feet are probably unnecessary; however, in regions where large volumes of sand are known to be transported during intense storms, longer weirs may be indicated. Other factors such as cost and protection of dredges in the deposition basin also influence the selection of weir length. Longer weir sections generally cost less because less materials are required to construct the relatively low weir section; however, longer weirs afford less wave protection to vessels navigating the channel and to a dredge in the deposition basin.

IX. POSTCONSTRUCTION PROFILE ADJUSTMENT

After construction of jetty systems at inlets, initial profile adjustments occur because of modification of tidal current patterns by the jetties. Before construction, spreading ebb tidal currents work to keep the ocean shoal offshore. The shoal exists in a state of dynamic equilibrium; waves which tend to move the shoal onshore are balanced by ebb currents. Jetty construction directs ebb currents into a narrow channel where they no longer act over the entire shoal. The shoals updrift and downdrift of the jetties, although no longer acted on by ebb currents, continue to be acted on by waves. Therefore, much of the sand stored in the shoal may move onshore under the influence of waves. Following construction, the sand formerly in this part of the shoal contributes to the fillets between the jetties and updrift and downdrift beaches. On the updrift side of the project, the additional sand may eventually find its way into the deposition basin and lead to increased dredging requirements for several years after completion of construction. The rate at which sand moves onshore depends on the wave climate, particularly wave conditions during the immediate postconstruction period. The quantity of sand stored in the offshore bar can be estimated by comparing preconstruction profiles which extend through the shoal area near the inlet with profiles taken updrift and downdrift of the inlet at a distance away from the influence of tidal currents. Profile changes near the weir at Murrells Inlet are shown in Figure 34. The two surveys plotted in the figure were taken about 1 year apart and show a decrease in the size of the offshore shoal. Figure 35 shows two profiles taken about 11,100 feet updrift (northeast) of Murrells Inlet at the same time as the profiles were taken in Figure 34. Changes in these updrift profiles are less than changes occuring at the inlet. The first survey shown in Figure 35 was taken about 7 months after construction started on the north jetty. Figure 36 shows the updrift profile superimposed on the profiles near the inlet. The area between the profiles represents the amount of sand stored offshore which can potentially be transported to the beach and deposition basin by wave action. Additional closely spaced profiles in the vicinity of the inlet may be used in conjunction with the updrift profile to determine the volume of sand contained in the bar.

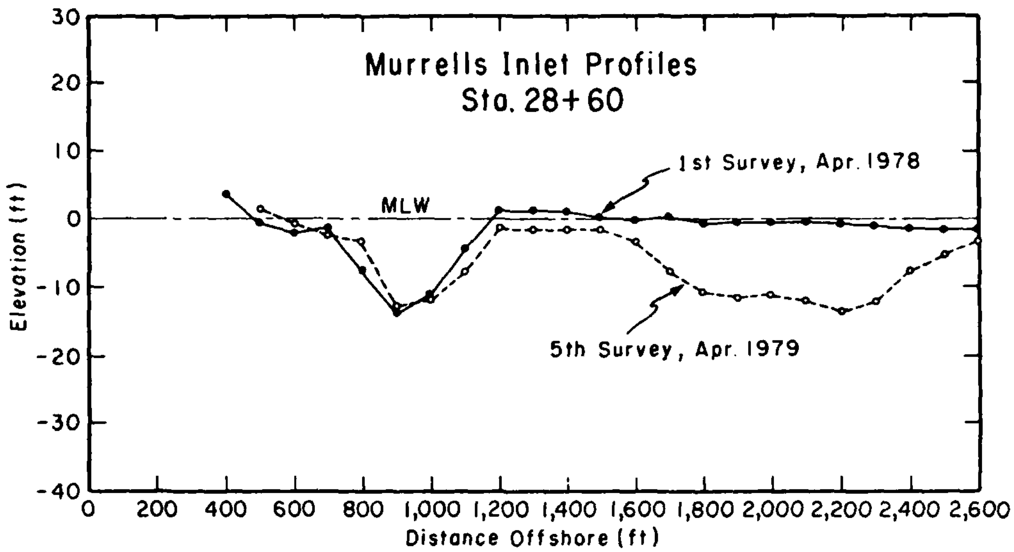

Figure 34. Changes in updrift profile near weir at Murrells Inlet, South Carolina.

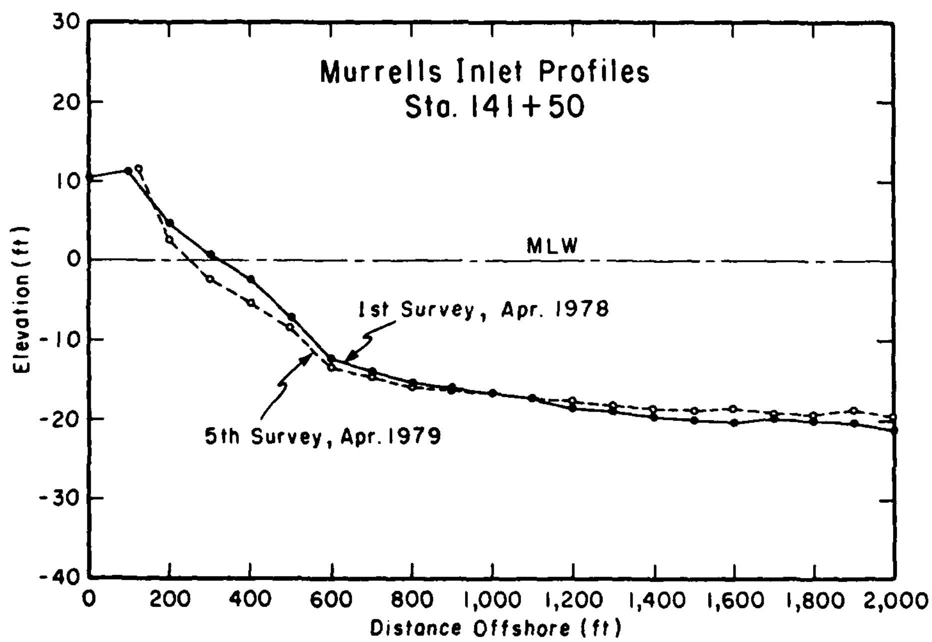

Figure 35. Beach profiles about 2 miles updrift of Murrells Inlet, South Carolina.

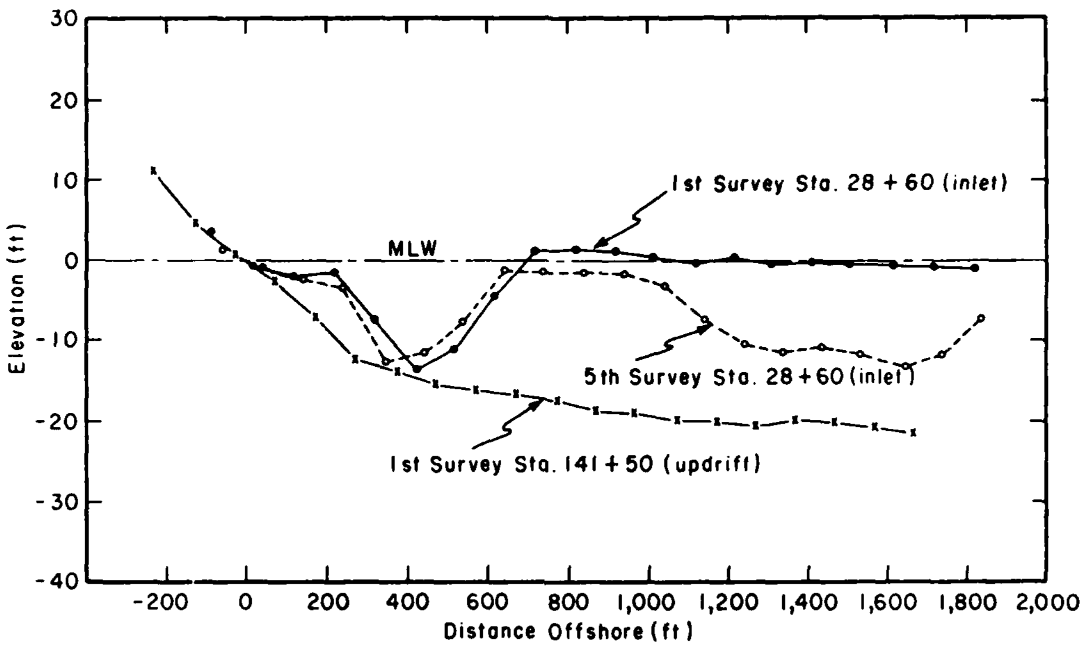

Figure 36. Updrift beach profile superimposed on the profiles near Murrells Inlet, South Carolina.

X. DESIGN OF UPDRIFT BEACH

Several methods are available to predict the realinement of the updrift beach after construction of a jetty complex. None of the methods are entirely satisfactory because they have not been verified by comparison with the behavior of actual beaches. Characteristics of the weir jetty that determine the response of the updrift beach are the length and orientation of the sandtight section landward of the weir, the length and elevation of the weir itself, and the orientation of the entire jetty system. The location of the landward end of the weir generally fixes the location of the shoreline at the weir and to a certain extent the planform of the updrift shoreline. The elevation and profile of the weir determine the beach profile near the jetty; this also influences the location and alinement of the updrift beach. The alinement of the overall jetty complex and the variability of direction of wave approach determine the amount of sheltering that the jetty affords to the updrift beach. The degree of sheltering or protection from wave action determines whether sand in the fillet adjacent to the jetty can be moved up the coast during periods of transport reversal.

The simplest method of predicting updrift beach response is to extrapolate the curvature of the existing shoreline into the region of the inlet. The landward end of the weir is then established at the intersection of the extrapolated shoreline and the proposed jetty alinement. Figure 37 shows an example shoreline which is updrift of Murrells Inlet. The use of the extrapolated shoreline for the design of the weir jetty at Murrells Inlet would have resulted in an extremely long sandtight section landward of the weir. Judgment must be used in interpreting the results of extrapolating the shoreline.

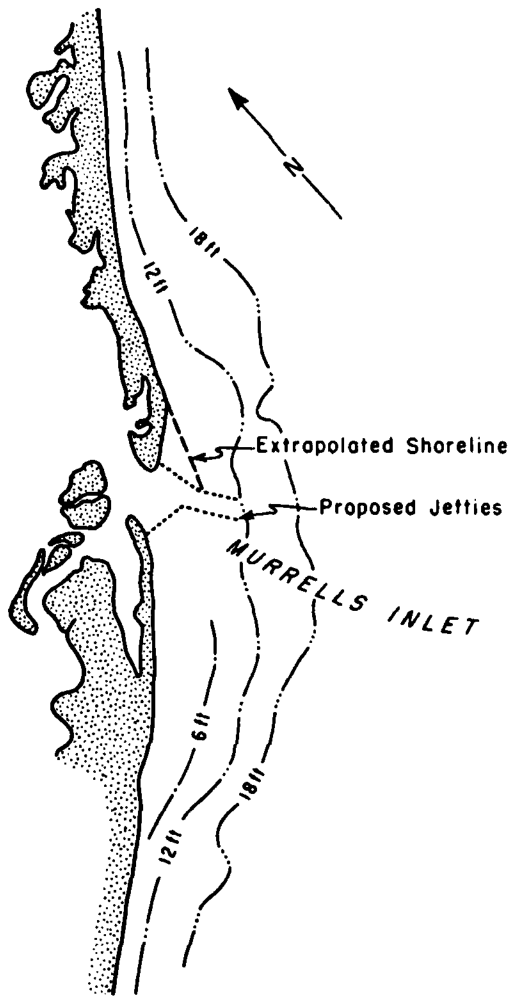

Figure 37. Extrapolation of shoreline updrift of Murrells Inlet.

Results at Murrells Inlet would have indicated a large volume of sand in storage updrift of the jetty. The presence of a groin field updrift of the inlet has had some influence on the shoreline and thus affects the extrapolation.

A second method of determining the shoreline near the jetties is to apply an analytical model to describe the shoreline evolution in response to incident wave conditions. The most often quoted model is that developed by Pelnard-Considere (1956). LeMehaute and Brebner (1961) and LeMehaute and Soldate (1977) describe the development of an expression for the updrift shoreline and for the amount of sand bypassing. Basically, the shoreline updrift of a groin or jetty as a function of time is given by the expression (variables are defined in Fig. 38)

$$y_s = \frac{\tan \alpha_b}{\sqrt{\pi}} \left[\sqrt{\frac{4t\varepsilon}{k}}\, e - \frac{x^2 k}{4t\varepsilon} - x\sqrt{\pi}\, E\left(\frac{x}{\sqrt{4t\varepsilon/k}}\right)\right] \tag{17}$$

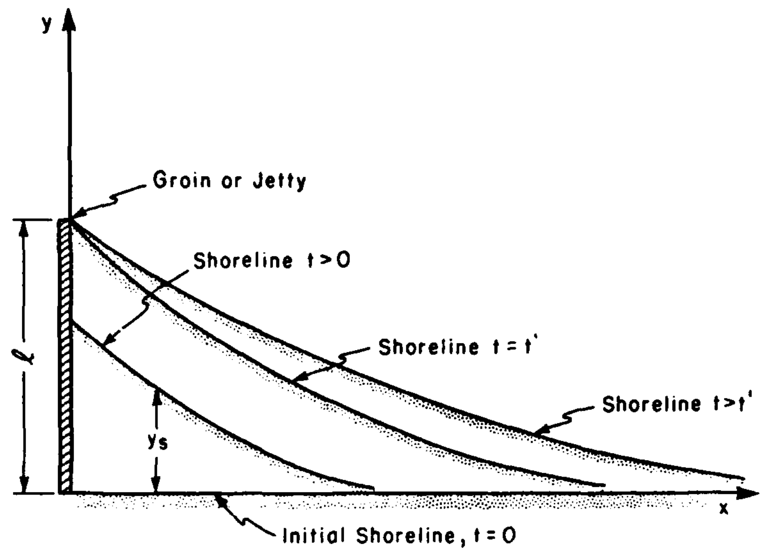

Figure 38. Definition sketch of the Pelnard-Considere model of shoreline evolution near a groin or jetty (after LeMehaute and Soldate, 1977).

where

y_s = distance the shoreline has moved seaward from its original straight configuration

x = distance updrift from the jetty or groin

t = time since construction of the jetty

α_b = angle the wave crest makes with the original shoreline

ε/k is a factor which depends on the offshore profile and wave conditions and was reported by Bakker (1968a, 1968b) to be approximately 4.3×10^6 cubic feet per year for an exposed location along the coast of Holland (The Netherlands). The variable $E(x/\sqrt{4t\varepsilon/k})$ is the error function complement of the argument $x/\sqrt{4t\varepsilon/k}$ and is tabulated in numerous handbooks of mathematical tables (e.g., Abramowitz and Stegun, 1964). The solution given by equation (17) describes the updrift shoreline until the time when the shoreline at the jetty reaches the end of the jetty; this time is given by

$$t' = \frac{\ell^2 \pi k}{4\varepsilon \tan^2 \alpha_b} \qquad (18)$$

After the time, $t \geq t'$, the jetty no longer retains all the sand moving into the fillet but bypasses a part of it. The amount of sand bypassed increases with time until the fillet is full (a straight shoreline built out to the end of the jetty), at which time all the sand approaching the structure is bypassed. After the sand begins to bypass but before the beach fillet is full, the shoreline is given by

$$y_s = \ell E\left(\frac{x}{\sqrt{4t\varepsilon/k}}\right) \tag{19}$$

where ℓ is the length of the jetty and $E(x/\sqrt{4t\varepsilon/k})$ is again the error function complement of the argument $x/\sqrt{4t\varepsilon/k}$. The solution for various times is shown in Figure 39. To describe the updrift beach behavior from $t = 0$ to $t = \infty$, the solutions given by equations (17) and (19) must be matched at the time when $y_s = \ell$; i.e., when $y_s = \ell$, $t = t'$ the shoreline from equation (17) must be matched with a shoreline described by equation (19). For $t > t'$ the solution is given by equation (19) with $t' = 0.62t$ substituted for the t in the equation.

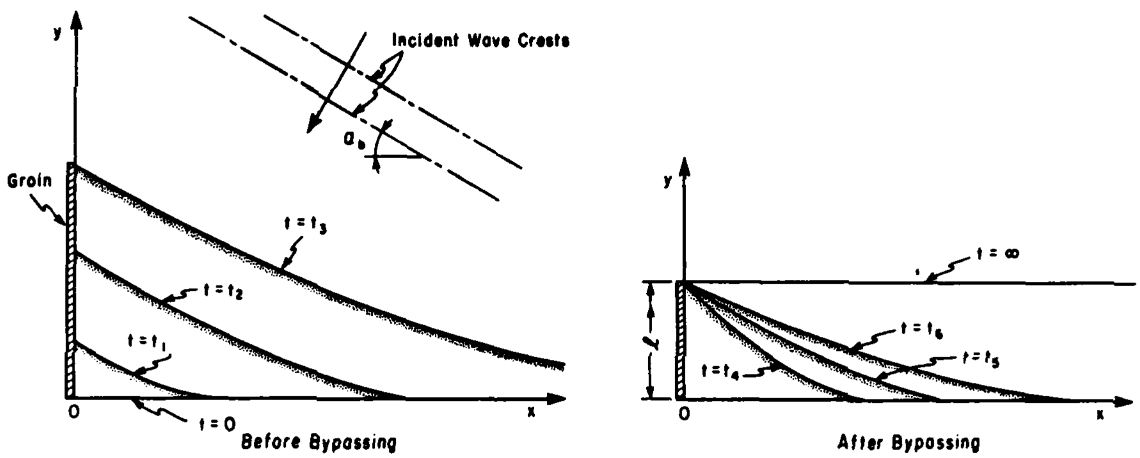

Figure 39. Shoreline evolution near a groin (after LeMehaute and Soldate, 1980).

The bypassing rate is given by the expression

$$\frac{Q_\ell}{Q_{\ell o}} = \left(1 - \frac{0.638}{\sqrt{(t/t') - 0.38}}\right) \tag{20}$$

and is shown in Figure 40. The quantity $Q_{\ell o}$ is the longshore transport that would occur along a straight beach and is the transport rate to which Q_ℓ is asymptotic as $t \to \infty$. The coefficients 0.638 and 0.38 in equation (20) were selected to match the solutions described by equations (17) and (19).

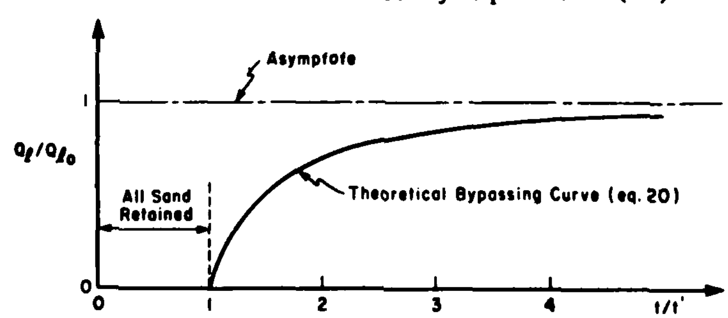

Figure 40. Sand bypassing around a groin (after LeMehaute and Soldate, 1980).

Application of the Pelnard-Considere (1956) solution to the design of the updrift beach at a weir jetty requires that the wave climate at the site be described by a single wave. It is assumed that the shoreline can be characterized as the beach planform that results from that wave. Also, since refraction is not considered, the final shoreline when $t \rightarrow \infty$ is straight; therefore, the shoreline at some arbitrary time must be chosen to describe the beach's response to jetty construction. At best, Pelnard-Considere allows the use of nearby shoreline geometry to be extrapolated or transposed to predict the shoreline response near a jetty.

The most promising method of predicting the behavior of the updrift shoreline is to use a numerical model to describe the sediment balance for numerous small lengths of shoreline. A number of such models have been developed (Perlin, 1978; LeMehaute and Soldate, 1980) with varying levels of success. The advantage of a numerical model is that the actual wave conditions at the site can be simulated and used in the model to generate the postproject shoreline. The difficulty in using these models is calibrating them for a site. The simple models do not describe the complex nearshore processes that determine the two-dimensional bathymetry near structures since only the one-dimensional empirical relationship is usually used between longshore sand transport and longshore wave energy flux.

Numerical models, with further development, show promise for predicting shoreline changes caused by jetty construction. Presently, these models are qualitative; however, if calibrated using nearby structures and shoreline changes, the models might be used to transpose results from adjacent sites to a weir-jetty site.

XI. APPLICATION OF HYDRAULIC MODELS TO THE DESIGN OF WEIR-JETTY SYSTEMS

Construction of a weir-jetty system involves a sizable investment of both time and money. For example, the weir-jetty system at Murrells Inlet was built at a cost of $11.4 million over a period of 3 years. Project construction was initiated in October 1977 and completed in August 1980. Because of the high cost of such projects, any design deficiencies will usually be expensive to correct and small design improvements may result in significant savings; therefore, the designer should use every method at his disposal to ensure an adequate, optimum design which satisfies both functional and structural requirements. A hydraulic model testing program for a proposed weir-jetty system should be the rule rather than the exception whenever a relatively large jetty project is conceived. Model tests can investigate the hydraulics of inlet modification, changes in sediment transport conditions caused by inlet modification, and the structural stability of proposed jetty cross sections. Although the results obtained from model tests are not always in total conformance with final behavior of the prototype, they can provide quantitative information on project performance. At the very least, a qualitative understanding of the effect of the project can be obtained. The level of confidence in model test results depends on the type of model and its purpose. A detailed presentation on coastal hydraulic models is given by Hudson, et al. (1979).

1. Hydraulics of Inlet Changes.

A number of factors in the design of a weir-jetty system can be considered in a three-dimensional, fixed-bed model of the inlet and proposed improvements. These include changes in hydraulic characteristics of the inlet-bay

system brought about by weir-jetty construction and, to a limited extent, changes in shoaling and deposition patterns. Hydraulic factors that can be quantified are the effects of jetty construction on the tidal prism, the tidal range in the bay, and the current patterns in the vicinity of the inlet. These factors are not peculiar to weir jetties but rather pertain to all jetty projects. If undistorted, with vertical and horizontal scales the same, a fixed-bed model may be used to study the effect of waves on circulation patterns, to estimate the level of wave protection afforded vessels navigating the channel, and to a dredge in the deposition basin. Additionally, any tendency for currents to move up against the jetty or weir structures can be observed. Design modifications may then be made and evaluated in the model that will preclude undermining the prototype structures. For relatively simple weir geometries such as sheet-pile weirs, the part of the tidal prism entering the inlet across the weir can be estimated from the model. How flow over the weir varies with time can be studied along with the relative magnitude of ebb and flood current velocities in the navigation channel.

2. Scour, Deposition, and Sediment Transport.

Scour, deposition, and sediment transport in proposed weir-jetty systems can be studied using either movable-bed models or fixed-bed models with tracer materials such as coal, plastic beads, etc., placed on the bottom. Movable-bed modeling technology has not developed to the point where reliable information on bottom changes can be obtained. The models are difficult to calibrate and verify, requiring large quantities of prototype data for this purpose. They are also expensive to operate and the cost of obtaining data for verification can be prohibitive. In some cases, even after careful calibration, the results may be at best qualitative.

The validity of results obtained from fixed-bed models with tracer materials used to simulate scour and deposition is quite limited. They can not model changes in bathymetry but can be used to identify areas where scour and deposition will probably occur. Any large-scale changes in bathymetry and their subsequent effect on additional scour and deposition are not considered. Scour and deposition patterns are detected by placing a uniform thin veneer of a tracer material on the fixed bed of the model. The model is then run and the redistribution of tracer observed. A tendency to scour is indicated by removal of tracer while accumulation of tracer is interpreted as a tendency to accrete. Some information can be obtained by putting a grid system on the model floor and quantifying the relative amounts of tracer accumulated or removed from each grid square and comparing this with the amount originally present in the square. For example, scour near the base of proposed jetties may indicate a potential problem with undermining of the structure. Currents moving up against the structure could lead to hazardous navigation conditions and currents moving through the deposition basin might reduce its effectiveness in trapping sand.

The efficiency of a weir in trapping sediment in longshore motion can be estimated by tracer in a fixed-bed model. Tracer material is injected into the model along the updrift beach and waves are allowed to move it alongshore to the structure. Generally, a part of the tracer is trapped in a fillet along the updrift beach; a part moves over the weir into the deposition area and some may move along the weir and updrift jetty into the navigation channel. Various jetty layouts and weir configurations may be evaluated and an optimum layout selected from a sand bypassing point of view. Such tests are only semiquantitative but may still provide information on relative effectiveness of various jetty layouts.

Seasonal and irregular temporary reversals of longshore sand transport which influence the updrift and downdrift shorelines can be studied using tracer which has been initially distributed along a model shoreline and noting its response to waves from various directions. The amount of sand in dead storage updrift of the updrift jetty can be estimated along with the active storage if the wave heights, directions, and durations selected for the testing program are reasonably characteristic of the project site. The tracer is used to simulate a beach adjacent to the proposed inlet structures, and waves approaching from the updrift direction are run to form a fillet. Subsequently, waves from the downdrift direction are run and changes in planform of the fillet observed. Usually, all of the tracer moved into the fillet by waves from the updrift direction can not be removed from the fillet by waves from the downdrift direction because of the diffraction shadow of the jetties. This sheltering effect can be evaluated from observations of the tracer fillet and its response to variations in the magnitude and direction of incident wave energy.

3. Jetty Structure Stability.

Since the cost of large rubble-jetty structures is high, any cost-savings that can be obtained by prudent design and economic optimization should be investigated. Stability tests of various structure cross sections should be pursued if there is any question regarding the structural performance of the jetties under the varying wave and water level conditions prevailing at a site. Generally, jetties are designed for waves with heights limited by shallow depths close to shore (see Sec. 7.12 of the SPM). The limiting design breaker height is thus a function of the maximum water levels that can occur at the jetty site. Maximum wave conditions will prevail during maximum water levels such as occur during hurricanes. If a rubble jetty is designed for given water level and breaker height, greater water levels and breaker heights will result in some damage to the structure. The level of damage and the ability of the jetty cross section to continue its protective function can be assessed in a model. Models of the jetty trunk and head should be subjected to the various conditions of water levels and wave heights characteristic of the site. The model can also be useful to look at the interlocking between the armor and first underlayer and for sizing the underlayers. Also, waves incident on jetties usually propagate roughly along the axis of the structure. Little information is available on rubble structure stability under such wave action. If indicated, stability tests using waves with an oblique angle of approach should be conducted.

The weir section of a jetty system will overtop during a significant part of the tidal cycle. If the section is built of rubble, unique stability problems might arise because of overtopping. Special consideration should be given to testing the weir cross section for stability under these conditions.

Other savings may often be achieved by varying the structure cross section to allow smaller armor units close to shore where breaker heights are lower. Also, the landwardmost part of a jetty may not be subjected to significant levels of wave action because of the accumulation of sand adjacent to the jetty. Decreasing water depths close to shore limit the wave heights to which the nearshore segments of a jetty are subjected; consequently, jetties may be designed for lower wave heights provided sufficient information on the post-construction beach profile adjacent to the jetty is available. Model testing of the various cross sections should be performed.

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